CITY OF EL CENTRO
WATER MASTER PLAN UPDATE

prepared for the

CITY OF EL CENTRO

September 1994

REGISTERED PROFESSIONAL ENGINEER
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EXP. DATE 3/31/97
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prepared by

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San Diego, California 92037

September 30, 1994

Mr. Danny Brammer, P.E.,
Director of Public Works/City Engineer
City of El Centro
1275 Main Street
El Centro, CA 92244

Re: Water Master Plan Update

Dear Mr. Brammer:

Attached are five copies and one set of reproducibles of the Water Master Plan Update for the City's water treatment and distribution system. The Master Plan identifies capital improvements to the City's water treatment and distribution systems to meet the water demands of existing and future development within the City.

Improvements at the treatment plant recommended for immediate implementation are addition of a spare filter and a standby filter backwash pump. Recommended improvements at the plant within the next five years are addition of a raw water storage pond, a fourth duty filter (by 1998), and upgrading of the capacity of the treated water transfer pump station to match the increased filtration capacity (by 1998).

Improvements to the distribution system presently needed include upgrading the standby pump at the water treatment plant pump station and installation of segments of water transmission pipelines to improve distribution. Within the next five years, installation of the third pump at the remote reservoir pump station (by 1999) will be necessary. Future installation of water transmission and distribution pipelines will depend on the extents of future development.

Please contact us if you have any questions regarding the Water Master Plan Update. We have appreciated the opportunity to work with the City in development of this Master Plan Update.

Very truly yours,

ENGINEERING-SCIENCE, INC.

Daniel R. Duprey, P.E.
Project Manager

DRD:rj

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FIGURES

TABLES

ABBREVIATIONS

CCI	Construction Cost Index
DHS	State Department of Health Services, Drinking Water Division
ENR	Engineering News Record
ES	Engineering-Science, Inc.
ft	feet or foot
gal	gallon
gpcd	gallons per capita per day
gph	gallons per hour
gpm	gallons per minute
gpm/sf	gallons per minute per square foot
hp	horsepower
IID	Imperial Irrigation District
LAFCO	Local Agency Formation Commission
MG	million gallons
mgd	million gallons per day
psi	pounds per square inch
psig	pounds per square inch gauge
sf	square feet
UFC	Uniform Fire Code
VFD	variable frequency drive
WTP	water treatment plant

EXECUTIVE SUMMARY

EXECUTIVE SUMMARY

SECTION 1 - INTRODUCTION

The Master Plan for the City's water treatment and distribution system was last updated in 1982. The City has since completed nearly all of the improvements to treatment and distribution facilities recommended in the 1982 Master Plan. This Master Plan Update has been prepared as a periodic update to: (1) provide a long-range water treatment and distribution system management program and planning tool; (2) identify capital improvements necessary to maintain reliable water service to existing users; and (3) identify capital improvements necessary to provide water service to future development within the City's ultimate service area.

SECTION 2 - STUDY AREA

The planning area of the City's 1990 General Plan was adopted as the study area for the Master Plan Update. The planning area is bounded by IID's Central Drain on the north, McCabe Road on the south, and Highway 111 on the east. This area extends beyond the City's present sphere of influence on the south and east, but does not extend to the potential limits of the revised sphere of influence east of Highway 111. The ultimate service area was assumed to be the existing City limits and the Phase I and Phase II planning areas. Areas within the General Plan planning area but not covered by the Phase I or Phase II areas were assumed to remain undeveloped. For the purposes of this Master Plan Update, a current population of 36,450 and an average annual growth rate of 3.15 percent over a 20 year planning period were established.

SECTION 3 - WATER DEMAND PROJECTIONS

Based on City records, current water demands are as follows:

Annual Average Daily Demand	7.5 mgd	206 gpcd
Maximum Month Average Daily Demand	10.6 mgd	291 gpcd
Maximum Day Demand	12.5 mgd	343 gpcd
Peak Hour Demand	19.4 mgd	22.2 gph

Water demand projections were based on the assumption that per capita consumption will remain the same and on the 3.15 percent annual growth rate. Future demands were therefore estimated by escalating current demand rates at 3.15 percent per year. Fire flow demand was established as 4,000 gpm at 20 psi residual pressure for a four hour duration. This is based on the requirements of the Uniform Fire Code, fire flow required for existing development, and on the assumption that new construction will include sprinkler systems which will allow reduction of requirements greater than 4,000 gpm.

SECTION 4 - EVALUATION OF EXISTING RAW WATER STORAGE AND FILTRATION CAPACITY

Raw Water Storage Capacity Evaluation

Raw water is delivered to the treatment plant from the Date Canal through a 42-inch pipeline and is pumped into two raw water storage reservoirs. An alternative raw water supply is available from Dahlia Lateral No. 1 during shutdown of the Date Canal. The raw water storage reservoirs are asphalt lined earthen levee ponds. Each reservoir is 440 ft wide by 580 ft long by 18 ft deep (to top of levee) and has a storage capacity of 30 MG (14 ft depth), for a total raw water storage volume of 60 MG.

At the present average daily demand during the summer months, the raw water storage reservoirs provide 5.7 days of storage. In the event of shutdown of the All American Canal, there would be no raw water supply available to the plant other than that stored in the reservoirs. For the purposes of this Master Plan Update, a minimum raw water storage requirement of five days of the average day demand in the peak use month was established. An additional 30 MG reservoir will be required by the year 1998 to maintain the five day storage requirement. A fourth 30 MG reservoir will be required by the year 2011.

Filtration Capacity Evaluation

There are three existing filters, each with 936 sf of filter surface area for a total filter surface area of 2,808 sf. The filters were originally designed as mono-media but were converted to dual media in the early 1980's. Although dual media filters can operate at filter loading rates of up to 6 gpm/sf, these filters are limited to a maximum loading rate of 3.5 gpm/sf due to original hydraulic design constraints. Based on the 3.5 gpm/sf rate, the existing filtration capacity is 14.2 mgd. Additional filtration capacity will be required to meet maximum day demand by the year 1998. It is proposed that two additional filters be provided for the ultimate treatment system, each with a capacity of 7.4 mgd. The second additional filter would be required by the year 2011.

Presently, with one of the three existing filters out of service during the summer months, the two remaining filters do not have adequate capacity to satisfy the daily demand. This situation has created difficulties in the past when repairs to a filter have been required during periods of high water demand. It is therefore recommended that a spare filter be provided as a standby to the existing filters.

Impacts to Clarification and Treated Water Transfer Pumping Capacity

Increasing the capacity of the filtration system will have impacts on other treatment facilities. The treated water transfer pump station, which pumps filtered water to the treatment plant storage tanks, will have to be upgraded in capacity when the additional duty filter is constructed (by 1998). Additional clarification capacity will be required by the year 2004.

Standby Backwash Pump

Backwashing of the filters is accomplished using one 8,000 gpm backwash pump. If the pump is taken out of service, a connection to the water distribution pump station is provided as a standby backwash water supply. However, use of connection interferes with operation of the distribution system and is not practical. The Department of Health Services requires that a standby backwash pump be provided.

SECTION 5 - EVALUATION OF TREATED WATER STORAGE, TRANSMISSION, AND DISTRIBUTION SYSTEM

Storage Capacity Evaluation

Treated water is stored in three above-ground steel tanks at the treatment plant site and one above-ground steel tank in the vicinity of La Brucherie Road and Barbara Worth Avenue. Two of the tanks at the plant site have storage capacity of 2.5 MG each and the third has a capacity of 5.0 MG for a total storage capacity at the plant site of 10 MG. The capacity of the remote tank is 5.0 MG and there is space available at the remote site for an additional tank with a capacity up to 7.5 MG.

The total treated water storage requirement is based on one day of average summer demand plus fire flow. The present treated water minimum storage requirement is therefore 11.6 MG, which is less than the existing treated water storage capacity of 15 MG. Additional storage will be required by the year 2003. At that time a 5.0 MG tank at the remote site is recommended. Ultimately, an additional 5 to 6 MG of storage capacity will be required by the year 2013. This capacity could either be provided by additional tanks at the treatment plant, or by tanks at a second remote site.

There are also two elevated storage tanks in the system: one at 3rd and Commercial with a capacity of 100,000 gal and one at 8th and Vine with a capacity of 250,000 gal. Completion of the recent improvements to the distribution system to increase the operating pressure to 60 psi has made these tanks obsolete and they were not evaluated as part of the distribution system.

Pumping Capacity Evaluation

There are two pump stations which pump into the water distribution system: one at the treatment plant and one at the remote reservoir site. The treatment plant pump station consists of three 4,000 gpm capacity, variable speed pumps and one constant speed standby pump and has a combined capacity of 12,000 gpm. The remote pump station has two 3,500 gpm variable speed pumps, one of which is considered a standby. The remote pump station is designed to accommodate a third 3,500 gpm pump. The criteria for pumping capacity is peak hour demand for the combined capacity of both pump stations and maximum day demand plus daily fire flow (4,000 gpm for four hours) for the treatment plant pump station. Both criteria must be considered because water pumped from the remote tank to meet peak hour demand must be replenished by pumping from the treatment plant.

Based on the demand projections, the third 3,500 gpm pump at the remote pump station will be required by the year 1999 to satisfy the peak hour demand. A fourth 4,000 gpm duty pump at the treatment plant will be required by the year 2003 to satisfy the maximum demand day criteria. Ultimately, a total additional combined plant and remote pumping capacity of 9,000 gpm will be required beginning in 2012 to meet the peak hour demand, at least 5,000 gpm of which will be required at the treatment plant to also meet maximum day demand.

Transmission and Distribution Pipelines Evaluation

The treated water transmission and distribution system was computer-modelled by digitizing information from the water distribution system atlas, as-built subdivision drawings, and topographical maps. The reservoirs and pump stations were included in the model. Demand was distributed throughout the model based on water meter data. The model was calibrated with pressure test data provided by the City, then run under various demand conditions.

The results of the modeling indicated that there are no significant deficiencies in the existing distribution system. Installation of five transmission and distribution pipeline segments is recommended to improve service to the south-west and south-east areas of the City. The recommended transmission main installations are: (1) 2,400 ft of 27-inch transmission main in La Brucherie Road from Wake Avenue to Ocotillo Drive across Interstate 8; (2) 1,300 ft of 18-inch transmission main parallel to the existing 18-inch line along Date Drain from Danenburg Road to Wake Avenue; (3) 2,500 ft of 24-inch transmission main parallel to the existing 18-inch line in Danenburg Road from the treatment plant to Date Drain; (4) 325 ft of 12-inch connection in Ross Avenue from 1st Street to 2nd Street; (5) 1,300 ft of 12-inch main in 4th Street from Ross Avenue to Hamilton Avenue. Pipeline segments 1 through 3 were recommended in the 1982 Master Plan Update and will provide a more direct service to the south-west and west areas of the City. Pipeline segments 4 and 5 will improve the distribution system looping in the south-east area.

The model was also used to develop a conceptual layout of the ultimate water distribution system. These future pipelines will be installed to provide service to future development in presently undeveloped areas within the City's sphere of influence.

SECTION 6 - SUMMARY OF RECOMMENDED IMPROVEMENTS AND COST ESTIMATES

Estimated costs and recommended phasing for improvements to the existing treatment, pumping, storage, and distribution facilities are summarized in Table ES.1. Table ES.2 shows an estimate of the additional transmission and distribution mains required for the ultimate service area.

Table ES.1

Summary of Recommended Improvements and Suggested Phasing

Fiscal Year	Recommended Improvement (year required)[a]	Estimated[b] Cost ($1,000)
1995/96	Standby filter backwash pump (1995)	$200
	WTP standby pump upgrade (1995)	$250
	La Brucherie Road 27-inch main	$490
	Date Drain/Danenburg Rd. parallel mains	$325
	Total 1995/96	$1,265
1996/97	Spare filter (1995)	$1,400
	7.4 mgd filter (1998)	$1,600
	Total 1996/97	$3,000
1997/98	30 MG raw water storage reservoir (1998)	$2,100
1998/99	3,500 gpm remote pump (1999)	$100
	Ross Avenue 12-inch connection	$20
	4th Street 12-inch main	$100
	Total 1998/99	$220
2003	5.0 MG remote storage tank (2003)	$1,000
2005	4,000 gpm WTP distribution pump (2005)	$250
2011	30 MG raw water storage reservoir (2011)	$2,100
	7.4 mgd filter (2011)	$1,600
	Total 2011	$3,700
2012	5,000 gpm WTP distribution pump (2012)	$700
2013	5.5 MG treated water storage (2013)	$1,500
2017	4,000 gpm distribution pump capacity	$500
	Total Improvements	$14,200

[a] Improvements to water treatment facilities address raw water storage and filtration system only.

[b] ENR Construction Cost Index 6510, April 1994.

Table ES.2

Estimated Costs for Future Water Distribution System Pipelines for Ultimate Service Area

Development Area	Diameter (inch)	Length (feet)	Unit[a] Cost ($/ft)	Total Cost ($1,000)
South	30	18,600	$110	$2,050
	27	1,500	$100	$150
	18	11,000	$75	$830
	12	47,700	$60	$2,860
West	24	14,600	$90	$1,310
	12	38,500	$60	$2,310
North	24	14,100	$90	$1,270
	18	7,300	$75	$550
	12	14,100	$60	$850
East	30	300	$110	$30
	27	8,000	$100	$800
	18	2,300	$75	$170
	12	42,600	$60	$2,560
Total for Future Water Distribution Pipelines				$15,740

[a] ENR Construction Cost Index 6510, April 1994.

SECTION 1

INTRODUCTION

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INTRODUCTION

BACKGROUND

The City of El Centro, incorporated in 1908, is located in the Imperial Valley in Imperial County. Although the City's economy was founded on agriculture, the largest current sectors in the economy are government and the wholesale/retail trade. El Centro has become a regional administrative and commercial center for the county. Between the mid-1940's and 1994, the City's population grew from roughly 11,000 to 36,450, representing a three-fold increase in five decades.

Planning for the City's water system has been undertaken in stages:

- A master plan for the City's water system was prepared by Design Sciences, Inc. in 1977 and addressed the existing water supply, treatment, storage, and distribution facilities.
- In 1982, Lyon Engineers, Inc. updated the water master plan (Reference 1) and recommended a number of system improvements, including a new treated water storage reservoir and pump station on the west side of the City.
- In 1988, Engineering-Science, Inc. (ES) prepared an evaluation of alternatives for the west side reservoir and pump station (Reference 2). ES subsequently designed the facilities, and construction was completed in 1993.

To complement ongoing planning efforts for the City and to accommodate future growth, the City has decided to update the water system master plan. In December 1993, the City authorized ES to update its water master plan.

1982 MASTER PLAN UPDATE

The 1982 Master Plan Update recommended the following future improvements to the water treatment and distribution system (completed improvements are so noted):

- Replace the 405 ft long, 24-inch diameter raw water pipeline in the alternate raw water supply system with a 36-inch pipeline by the year 1988. (A parallel 36-inch pipeline was installed.)
- Replace the 2,070 ft long concrete-lined raw water supply ditch with a 48-inch pipeline from the Dahlia Lateral No. 1, Gate 18 to the outlet works of the existing ditch by the year 1994. (This replacement was completed.)
- Stabilize the banks of the raw water storage reservoirs and modify the reservoir piping to improve circulation. (The storage reservoir banks were stabilized with concrete in 1989.)
- Upgrade the distribution pump station at the treatment plant. (This work was completed in 1993.)

- Provide an altitude valve for the 3rd Street tank to allow the system to operate off the 8th Street tank without overflowing the 3rd Street tank. (An altitude valve is not required with the 60 psi system.)
- Construct a 5.0 million gallon (MG) remote storage reservoir with pump station and chlorination station for additional fire flow and 24-hour domestic reserve and for emergency alternate supply. (These facilities were made operational in 1993.)
- Construct an 18-inch feeder in La Brucherie Road extending from Hamilton Avenue to Lincoln Avenue. (This feeder was installed and presently extends north of Bradshaw Avenue.)
- Replace 350 ft of 4-inch cast iron line in 12th Street between Main Street and Broadway with 12-inch pipe. (This replacement was completed.)
- Replace 75 ft of 6-inch line at the intersection of 9th Street and Main Street with 12-inch pipe. (This replacement was completed.)
- Install an 18-inch main parallel to the existing 18-inch main from the water treatment plant to Imperial Avenue.
- From the parallel 18-inch line from the water treatment plant to Imperial Avenue, install a 24-inch line to La Brucherie Road, then an 18-inch line extending north in La Brucherie Road to Ocotillo Drive. (An 18-inch line was installed from Imperial Avenue to La Brucherie Road.)
- Replace the 12-inch cast iron line in Commercial Avenue with an 18-inch line from Main Street (actually 5th Street) to the 3rd Street elevated tank. (This replacement was completed.)
- Install a 12-inch line in Hamilton Avenue from La Brucherie Road to Waterman Avenue, south in Waterman to Vine Street, then east in Vine to Imperial Avenue. (This line was installed straight through on Hamilton.)
- Install a 12-inch line from the existing 18-inch line at 1st Street and Orange Avenue to Dogwood Road.
- Install an 18-inch line in Wake Avenue from 4th Street to 5th Street. (An 18-inch line was installed in Wake Avenue from 4th Street to 8th Street.)
- Continue replacement of cast iron water mains. (Most of the cast iron water mains have been replaced.)

The 1982 Master Plan also described transmission and distribution projects in progress at the time which have since been completed including: (1) an 18-inch feeder in Wake; (2) an 18-inch connection from 4th and Driftwood to 3rd and Ross, then a 12-inch line to the 12-inch in 1st (the connection to 1st was not made); (3) replacement of the 8-inch line in 3rd from Ross to Hamilton; (4) an 18-inch loop in Commercial from 3rd to 1st then south in 1st to Orange; and (5) a 12-inch loop in Commercial from First to Dogwood, then south in Dogwood to Holt.

1988 ALTERNATIVES STUDY FOR PROPOSED STEEL RESERVOIR AND PUMP STATION

As noted above, the 1982 Master Plan recommended that a treated water reservoir and pump station be constructed on the west side of the City. In 1988, an alternatives study for the remote facilities and for boosting the system operating pressure from 40 psi to 60 psi was conducted. The alternatives study recommended a 5 MG steel tank near La Brucherie Road and Barbara Worth Avenue, with variable speed pumping for both the remote pump station and the treatment plant pump station.

WATER SYSTEM IMPROVEMENTS PROJECT

In 1993, the City placed into operation the 5 MG remote storage reservoir and pump station as recommended in the 1982 Master Plan Update and 1988 Alternatives Study. The Water System Improvements Project also replaced three of the pumps at the treatment plant with variable speed pumps. The project revised operation of the system from floating on the elevated storage tanks to a completely pumped system. The modified treatment plant pump station and the new remote pump station are capable of pumping the design demand with the distribution system pressure at approximately 60 psi. Completion of the project and conversion to completely pumped operation has allowed the system operating pressure to be increased from around 40 psi (as limited by the height of the elevated tanks) to 60 psi at the 3rd Street elevated tank site.

1990 GENERAL PLAN

The current General Plan for the City was prepared in 1990 (Reference 3). The General Plan recommended a land use program for future expansion of the City in two phases of development. Phase I would occur in already settled areas and in undeveloped areas adjacent to existing development. Once development within Phase I approaches capacity, development would be permitted in contiguous areas during Phase II by City council action.

This phased growth management program is presently being re-evaluated based on current development plans. One of the factors affecting the ability to accommodate and manage new growth is the capability of the water system to satisfy demands from the proposed development. Updating of the water master plan will provide information to planners regarding the timing and cost of improvements necessary to accommodate future development.

REPORT OBJECTIVES

The primary objectives of this report are: (1) to provide a long-range water treatment and distribution system management program and planning tool; (2) to identify capital improvements necessary to maintain water service to existing users; and (3) to identify capital improvements necessary to provide water service to future development within the City's ultimate service area.

This report provides a comprehensive review and evaluation of the City's water storage and distribution facility requirements. Both current and projected ultimate development conditions are considered. Ultimate water demands were estimated using the land-use element of the City's General Plan. The raw water storage and filtration components of the water treatment plant were evaluated in terms of current and future capacity needs. Computerized analyses of the treated water storage and distribution system were performed for both present and ultimate demand conditions. Using the results of the computer analyses, requirements for new facilities have been identified. Associated costs for new facilities are summarized in tabular format along with approximate dates for required improvements.

SCOPE OF WORK

The scope of work for this Water Master Plan Update included the following seven major tasks:

- **Task 1 - Data Collection.** Collection of existing facility data, land use data, and water demand data.
- **Task 2 - Hydraulic Model Development.** Development of a computer-based network model to evaluate the performance of the distribution system and its deficiencies, and calibration of the network model using pressure data at different steady state demand conditions.
- **Task 3 - Planning Criteria Development.** Development of planning criteria for evaluation of the existing distribution system and assessment of required future improvements.
- **Task 4 - Existing System Analysis.** Determination of the deficiencies of the existing storage and distribution system based on the hydraulic network model and the adopted planning criteria.
- **Task 5 - Future System Improvements.** Identification of required storage and distribution system improvements for development anticipated within five years and for ultimate development conditions anticipated by the General Plan and Phased Growth Areas. Review of the capacities of the water treatment plant raw water storage reservoirs and filters and development of recommended modifications to these facilities to meet demands.
- **Task 6 - Service Area Plan.** Preparation of a Service Area Plan for the next 20 years in five year increments in accordance with Imperial Local Agency Formation Commission (LAFCO) requirements. (The Service Area Plan has been prepared as a separate document.)
- **Task 7 - Master Plan Report.** Preparation of a master plan report to document the results of the water system analyses.

SECTION 2

STUDY AREA

SECTION 2

STUDY AREA

The City of El Centro is located in the Imperial Valley in Imperial County, California, adjacent to Interstate 8. The City provides potable water treatment and distribution service within the City limits and will provide potable water service to future annexations. This section defines the Study Area for water master planning purposes and presents information regarding the Study Area population and land use.

STUDY AREA

The Imperial Local Agency Formation Commission (LAFCO) is responsible for establishing the service area boundaries for public agencies in Imperial County. The potential ultimate boundary for an agency is referred to as its "sphere of influence." The City's sphere of influence, established by LAFCO in 1977, covers a total area of 11,568 acres which includes the incorporated area of the City (approximately 4,200 acres) and adjacent unincorporated land (approximately 7,400 acres). Unincorporated portions of the sphere of influence may eventually be annexed to the City. The sphere of influence is bounded by:

- Imperial Irrigation District's Central Drain on the north
- Austin Road on the west
- State Highway 111 on the east
- Interstate 8, Danenberg Road, the City Water Treatment Plant, and Wake Avenue on the south

The City's 1990 General Plan covered areas to the south beyond the sphere of influence. The southern boundary for the General Plan planning area extended to McCabe Road.

The City is preparing to submit a request to LAFCO to extend its sphere of influence. On the south, the revised sphere of influence would extend to McCabe Road consistent with the General Plan planning area, and on the east, it would extend to the section line east of Highway 111. The 2,300-ft expansion east of Route 111 was not included in the 1990 General Plan but is potentially planned as a transportation corridor. The proposed sphere of influence covers an area of 17,500 acres.

The planning area of the 1990 General Plan is adopted as the Study Area for this report and is assumed to correspond to the potential ultimate water service area. Expansion of the service area to the east to incorporate the land east of Route 111 not previously considered to be within the City's planning area should be evaluated if the revised sphere of influence is adopted.

Figure 2.1 shows the present City limits, the existing sphere of influence, the 1990 General Plan area, and the Study Area.

LAND USE

Existing Land Use

Existing development within the incorporated portion of the Study Area is comprised of residential, commercial, industrial, and agricultural land uses. Although about 80 percent of the incorporated area is presently developed, only about 30 percent of the overall Study Area is developed. Existing land use in the Study Area may be characterized as follows:

- The Central Business District includes the major civic uses, government office buildings, and older commercial uses.
- Industrial development is concentrated in the east and southeast portions of the City.
- Multi-family residential developments are found immediately north and south of the Central Business District, west of Imperial Avenue adjacent to Valley Plaza, and north of Adams Avenue.
- Single-family residential developments are located in the southern, southwestern, northern and northwestern portions of the City.

Most of the recent development has occurred on the south and west sides of the City, near Interstate 8. In addition, there has been some recent residential and industrial growth in the southeastern section of the City and some recent commercial growth along Interstate 8. Most undeveloped land in the Study Area is located on the south side of Interstate 8 and east of the present city limits.

Outside the City boundaries, agricultural land use predominates, although new residential development is under construction at the western boundary of the City. Mixed land uses (older commercial, industrial, and residential developments) are found adjacent to the northern and eastern portions of the City. Agricultural uses predominate south of Interstate 8 on the City's southern border.

Detailed Land Use Plan

The City is currently conducting an update of the 1990 General Plan, including reconsideration of the Phased Growth Plan. For the purposes of this study, a detailed land use map was developed based on the land use element of the 1990 General Plan and on the current status of the General Plan Update (Reference 4). The land use map developed for this Master Plan is shown on Figure 2.2. Land use designations in the detailed land use plan of Figure 2.2 are the same as those used in the 1990 General Plan.

Figure 2.1 Study Area

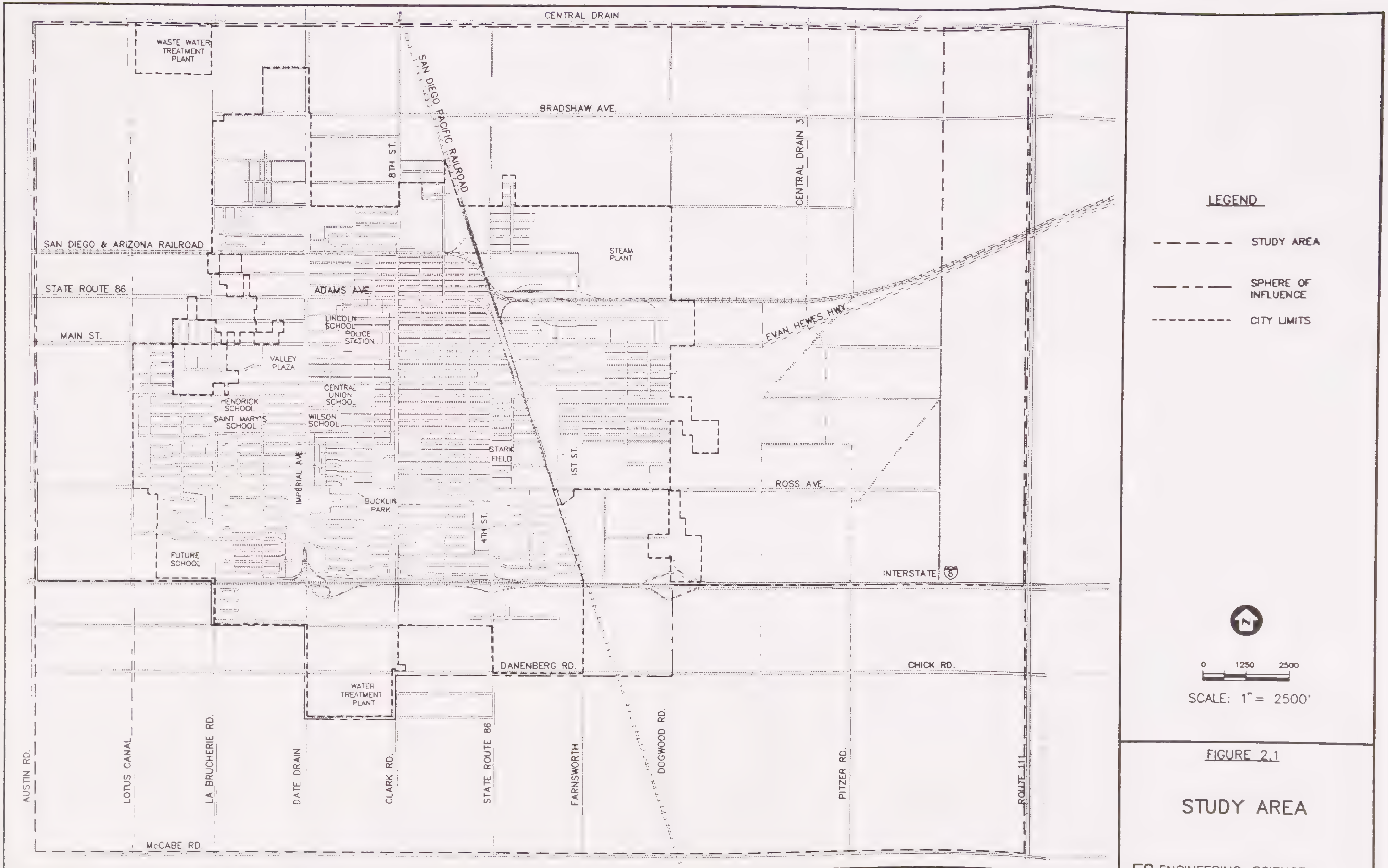

FIGURE 2.1

STUDY AREA

ES ENGINEERING-SCIENCE

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Figure 2.2 - General Plan Land Use Areas

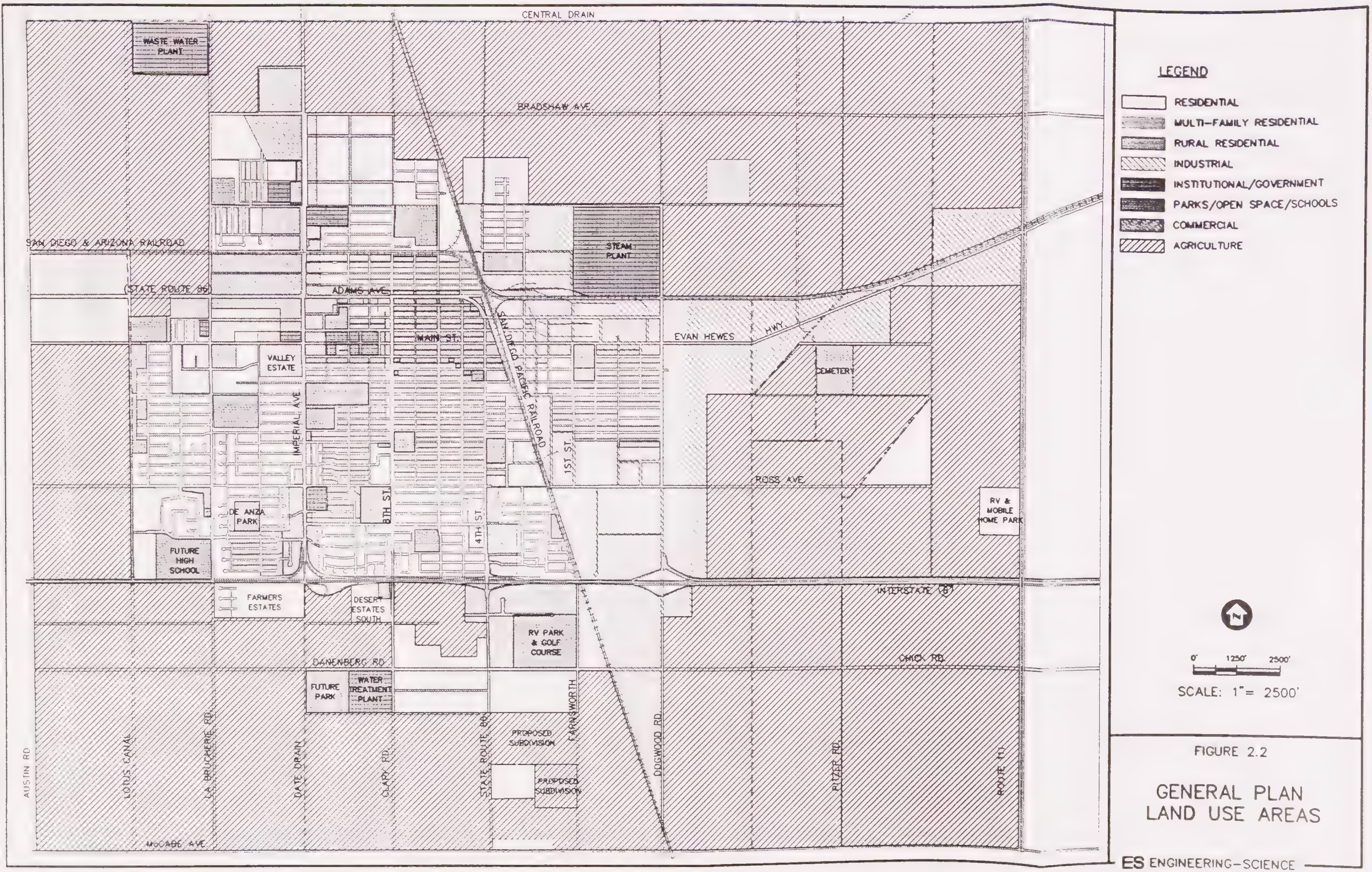

FIGURE 2.2

GENERAL PLAN LAND USE AREAS

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Phased Growth Plan

In the 1990 General Plan, it was proposed that expansion take place in two phases:

- Phase I expansion would occur in already developed areas or in undeveloped areas that are adjacent to existing development. Phase I would accommodate 31,750 additional people.
- As development in Phase I approaches capacity, expansion into contiguous areas (Phase II) would be permitted following appropriate City Council action. Phase II would accommodate 19,700 additional people.

Coupled with limited development and redevelopment within the incorporated portion of the Study Area (1,440 additional people), the Phased Growth Plan would allow an ultimate population increase of 52,890 within the Study Area. Figure 2.3 shows the Phase I and Phase II development areas.

Proposed Annexations

At the time of preparation of this Master Plan, the City was considering annexation of five areas referred to as the Northwest, West, Southwest, South, and East annexation areas. The total area proposed to be annexed is 2,005 acres. Water service to most of these areas is presently limited to non-potable water supplied by IID. Some of the areas are adjacent to existing City water mains. However, the majority of the existing development within these areas is not connected to the system. Water rates for properties outside of the City are higher than rates for properties within the City. The proposed annexation areas are shown on Figure 2.3 (Reference 5).

POPULATION

Historic Population

Historic population data and average annual growth rates for the City are summarized in Table 2.1.

Population Growth in the Study Area

The 1990 General Plan estimated the ultimate population in the Study Area as follows:

Area	Population
1990 City Population	31,154
Additional Population:	
Within Existing City	1,440
Phase I Growth	31,750
Phase II Growth	19,700
Ultimate Population	84,044

Figure 2.3 - General Plan Phased Growth Areas

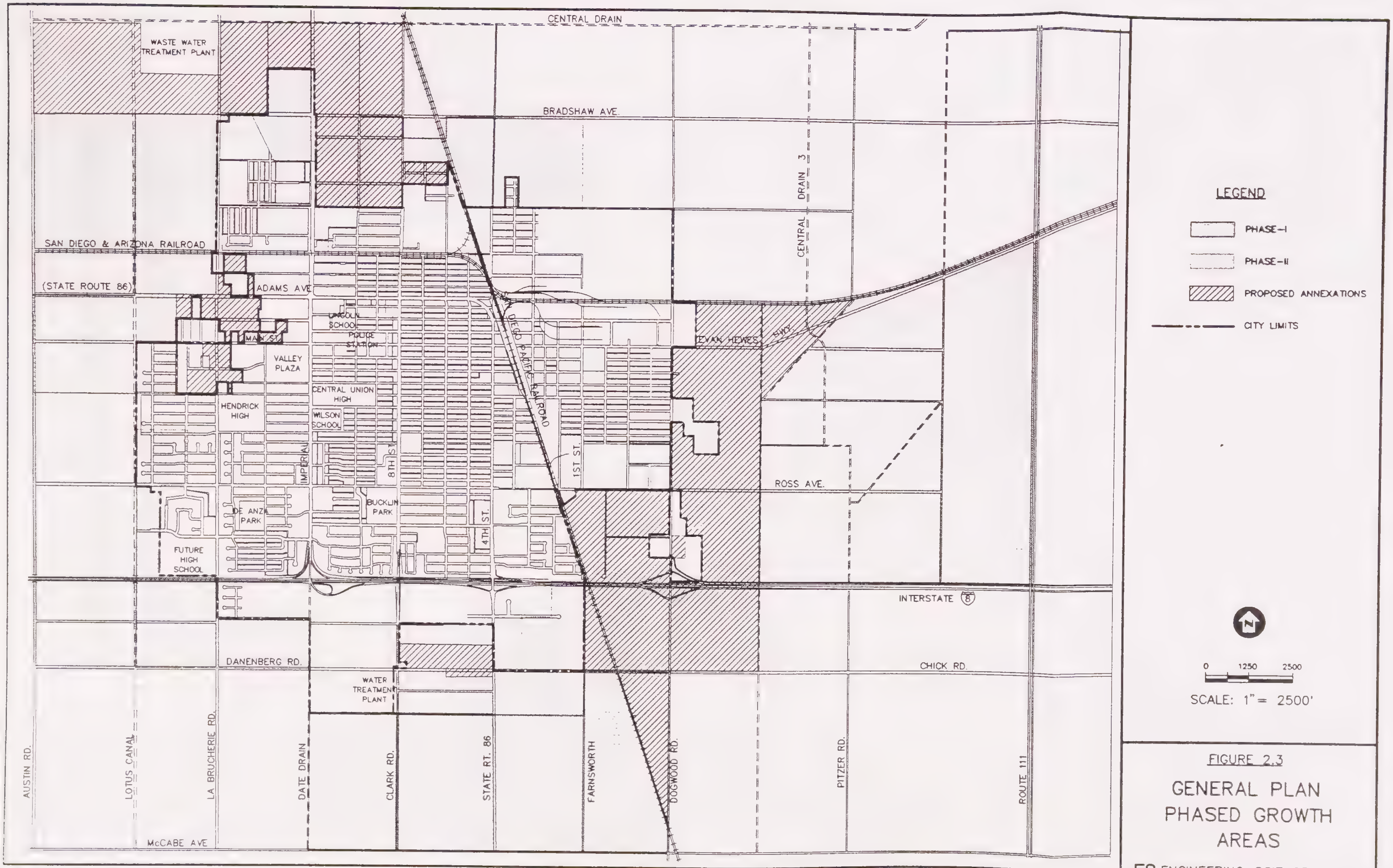

FIGURE 2.3

GENERAL PLAN PHASED GROWTH AREAS

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Table 2.1

City of El Centro Historic Population

Year	Population	Average Annual Growth Rate (%/year)
1960	16,811	
1970	19,272	1.37
1980	23,996	2.22
1990	31,154	2.64
1994	36,450	5.37

Source: References 3, 6

The 1990 General Plan indicated a wide variation in the future average annual population growth rate in the Study Area between 1990 and 2010, from a low rate between 0 and 1 percent to a high rate of 5.5 percent. Table 2.2 indicates the year that ultimate development would be attained assuming that these average growth rates are maintained.

Table 2.2

Population Growth Rate Range in the Study Area

Growth Scenario	Average Annual Growth Rate (%/year)	Year that Ultimate Popu-Population is Reached
No or slow growth	1.00	2078
Moderate growth	2.00	2037
Historical average growth	3.15	2021
Accelerated growth	3.80	2017
Explosive growth	5.50	2010

Note: Growth rate range defined in Reference 3

The Southern California Association of Governments projected the growth rate for El Centro at 3.15 percent based on historical growth patterns (Reference 3). For the purposes of this Master Plan, a growth rate of 3.15 percent and a 20 year planning period have been adopted. Based on the present estimated population 36,450 (Reference 6) and the 3.15 projected growth rate, the projected population of the City in 5 year increments for use in water system master planning is as follows:

Year	Projected Population
1994	36,450
1999	42,600
2004	49,700
2009	58,000
2014	67,800

SECTION 3

WATER DEMAND PROJECTIONS

SECTION 3

WATER DEMAND PROJECTIONS

CURRENT WATER DEMANDS AND CONSUMPTION

Water demand projections are necessary to establish design parameters used for sizing water treatment, storage, and distribution facilities. Demand projections are based on current usage and projected population growth. The following current demand criteria in million gallons per day (mgd) and gallons per minute (gpm) were established from treated water pumping data over the last two years as provided by the City:

Annual Average Daily Demand	7.5 mgd	5,200 gpm
Maximum Month Average Daily Demand (July and August)	10.6 mgd	7,400 gpm
Minimum Month Average Daily Demand (December and January)	4.3 mgd	3,000 gpm
Maximum Day Demand	12.5 mgd	8,700 gpm
Minimum Day Demand	3.6 mgd	2,500 gpm
Maximum Day Peak Hour Demand	19.4 mgd	13,500 gpm
Minimum Day Peak Hour Demand	10.8 mgd	7,500 gpm
Maximum Day Minimum Hour Demand	7.2 mgd	5,000 gpm
Minimum Day Minimum Hour Demand	2.2 mgd	1,500 gpm

Values for daily demand in gpm represent the average gpm rate over the day. Values for hourly demand in gpm represent the average gpm rate over the hour. Values for hourly demand in mgd correspond to the daily rate at the continuous gpm value.

PER CAPITA CONSUMPTION

Based on an estimated current population of 36,450 (refer to Section 2) and on the flow criteria listed above, per capita water use rates in gallons per capita per day (gpcd) are as follows:

Annual Average Use	206 gpcd
Maximum Month Average Use	291 gpcd
Minimum Month Average Use	118 gpcd
Maximum Day Use	343 gpcd
Minimum Day Use	99 gpcd
Peak Hour Use	22.2 gph

Per capita use rates have declined since 1970 when the estimate annual average use was 306 gpcd (Reference 1) to the current consumption rate of 206 gpcd. Generally, consumption has declined as water rates have increased and as a result of conservation. It is difficult to project future per capita consumption patterns, particularly since the distribution system pressure has recently increased from 40 psi to 60 psi and because an increase in water fees is pending at this time. For the purposes of this Master Plan Update, it is assumed that per capita consumption will remain at current rates, and that changes in demand will be solely a function of population changes.

PROJECTED WATER DEMANDS

Based on current demand and per capita consumption as described above and projected population growth as discussed in Section 2, projected water demands in five year increments are as listed in Table 3.1.

Table 3.1

Projected Water Demands

Year	Population[a]	Average Day (mgd)	Maximum Day (mgd)	Peak Hour (gpm)
1994	36,450	7.5	12.5	13,500
1999	42,600	8.8	14.6	15,800
2004	49,700	10.2	17.0	18,400
2009	58,000	11.9	19.9	21,500
2014	67,800	14.0	23.2	25,100

[a] Refer to Section 2.

Apart from two proposed subdivisions south of Interstate 8, there are no specific land use plans for undeveloped areas within the ultimate service area (Reference 4). The current General Plan land uses shown on Figure 2.2 do not show the full development potential of the area as described in the 1990 General Plan. For the purposes of estimating water demand for future development areas, the following assumptions were made:

- The future water service area will consist of existing incorporated areas, currently proposed annexation areas, and the Phase I and Phase II planning areas.

- Areas within the sphere of influence but not included in the ultimate service area will remain agricultural and will not require potable water service.
- The population density for Phase I areas will be the total projected Phase I population of 31,750 (Reference 3) divided by the estimated Phase I area of 2,700 acres, or 12 persons per acre.
- The population density for Phase II areas will be the total projected Phase II population of 19,700 (Reference 3) divided by the estimated Phase II area of 3,700 acres, or 6 persons per acre.
- Treated water demands for Phase I and Phase II areas will be based on the current per capita values described above.
- Areas currently proposed for annexation but not within Phase I or Phase II will be developed at the Phase I density.
- Undeveloped areas within the present City limits will be developed in accordance with the densities and per capita demands described above for existing development.

Based on the above assumptions, ultimate treated water demands will be as shown in Table 3.2.

FIRE FLOW DEMAND

The City has adopted the 1991 Uniform Fire Code (UFC) (Reference 7). The UFC sets fire flow requirements for buildings. The UFC defines fire flow as the flow rate of a water supply, measured at 20 psi residual pressure, that is available for fire fighting. Fire flow requirements may be modified upward or downward by the fire chief for special circumstances.

The UFC fire flow requirement for one- and two-family dwellings with a floor area less than 3,600 sf is 1,000 gpm for a duration of 2 hours. For one- and two-family dwellings with floor areas greater than 3,600 sf and for all other buildings, the UFC establishes fire flow and flow duration based on type of construction (per the Uniform Building Code) and on floor space area. The UFC required fire flows and flow durations range from a minimum of 1,500 gpm for two hours to a maximum of 8,000 gpm for four hours. The UFC allows a reduction in required fire flow of up to 75 percent, as approved by the chief, when the building is provided with an approved automatic sprinkler system; the resulting fire flow, however, shall not be less than 1,500 gpm.

The Fire Department reports that the current maximum fire flow requirement within the City is for the Costco Retail Warehouse. The UFC requirement for this building is 8,000 gpm for 4 hours, with a 50 percent reduction in flow rate for automatic sprinklers, for a fire flow of 4,000 gpm for 4 hours (Reference 8). Fire flow requirements from other recent studies for new construction have not exceeded 3,000 gpm.

Table 3.2

Projected Ultimate Water Demands

Parameter	Total Current	Phase I	Phase II	Ultimate
Estimated Population	36,450	27,894 [a]	19,700	84,044
Annual Average Daily Demand (mgd)	7.5	5.7	4.1	17.3
Maximum Month Average Daily Demand (mgd)	10.6	8.1	5.7	24.4
Minimum Month Average Daily Demand (mgd)	4.3	3.3	2.3	9.9
Maximum Day Demand (mgd)	12.5	9.6	6.8	28.9
Minimum Day Demand (mgd)	3.6	2.8	2.0	8.4
Maximum Day Peak Hour Demand (gpm)	13,500	10,300	7,300	31,100
Minimum Day Peak Hour Demand (gpm)	7,500	5,700	4,100	17,300
Maximum Day Minimum Hour Demand (gpm)	5,000	3,800	2,700	11,500
Minimum Day Minimum Hour Demand (gpm)	1,500	1,100	800	3,400

[a] Reduced from General Plan to account for growth since 1990.

The 1982 Master Plan Update projected a required fire flow of 5,000 gpm for 5 hours for the year 1996 and beyond. This estimate was based on the 1977 Master Plan which reportedly used, in part, determinations from a fire insurance classification in 1976 by the Fire Insurance Services Office. This type of city-wide fire insurance classification is for commercial buildings with fire flow requirements of 3,500 gpm or less. Commercial properties with fire flow requirements greater than 3,500 gpm are evaluated individually. Fire flows used in property insurance calculations are not intended to predict the maximum amount of water required for fire fighting.

Fire flow requirements for this Master Plan Update are based on the UFC and the current maximum requirement of 4,000 gpm at 20 psi residual pressure for four hours. This is based on the assumption that new construction will include automatic sprinkler systems which will allow reduction of requirements greater than 4,000 gpm to a maximum of 4,000 gpm. It is also based on the fact that sprinkler flow is normally localized to the fire area and does not require significant flow rates in addition to the fire flow rate.

SECTION 4

EVALUATION OF EXISTING RAW WATER STORAGE AND FILTRATION CAPACITY

SECTION 4

EVALUATION OF EXISTING RAW WATER STORAGE AND FILTRATION CAPACITIES

As discussed in Section 1, the scope of work for this Master Plan Update includes evaluation of the capacity of the raw water storage and filtration components of the City's water treatment plant. These treatment components have been identified by the City as potential limits to the overall capacity rating of the plant. It is anticipated by the City that the raw water storage reservoirs and filters will require expansion to meet treated water demands in the near future. This section presents a general description of the existing treatment facilities and evaluations of the capacities of the raw water storage reservoirs and the filtration system.

WATER SUPPLY

Water for the Imperial Valley comes from the Colorado River via the All American Canal. An extensive canal system operated and maintained by the Imperial Irrigation District (IID) distributes the water from the All American Canal throughout the Imperial Valley. Raw water is conveyed to the City's water treatment plant from the Date Canal, with the Dahlia Lateral No. 1 available as a back-up raw water source.

DESCRIPTION OF EXISTING TREATMENT FACILITIES

The existing water treatment facilities were constructed in the late 1950's, expanded in the early 1980's, and presently consist of the following:

- A 42-inch diameter raw water supply pipeline from the IID Date Canal, Gate 20-A on the east side of the plant to the raw water pump well, with a capacity of 20.7 mgd.
- An alternate raw water supply line consisting of a 2,070 ft long 48-inch diameter pipeline from the IID Dahlia Lateral No. 1, Gate 18 to two parallel 405 ft long, 24-inch and 36-inch diameter pipelines to the raw water pump well. This alternate supply line has a delivery capacity of 22.5 mgd.
- A raw water pump station consisting of four pumps capable of pumping a total of 24.1 mgd into the storage reservoirs, and also able to pump raw water out of the storage basins and into the treatment plant at 24.1 mgd in the event that both canal supplies were shut-off.
- Two asphalt-lined, earthen-levee raw water storage reservoirs, with a storage capacity of 30 MG each for a total capacity of 60 MG.
- Two circular solids contact clarifiers, each with a capacity of 8.5 mgd for a total capacity of 17.0 mgd.

- Three anthracite and sand dual media gravity filters with a total rated capacity of 20.2 mgd based on a filter loading rate of 5 gpm/sf. (As discussed below, the actual maximum loading rate is less than 5 gpm/sf due to hydraulic limitations of the filters.)
- A filter backwash pump with a rated capacity of 8,000 gpm, a backwash sump, and a backwash supernatant return pump.
- Treated water transfer pump station comprised of three vertical turbine pumps each having a capacity of 7.2 mgd for a total capacity of 21.6 mgd to pump treated water to storage.
- Liquid alum storage and feed facilities.
- Liquid chlorine storage (one ton cylinders) and gaseous chlorine feed (no evaporators) facilities.

The actual capacities of the water treatment processes are as follows:

• Raw water supply (Date Canal)	20.7 mgd	
• Raw water pumping	24.1 mgd	(no standby)
• Clarification	17.0 mgd	
• Filtration	14.2 mgd	(at 3.5 gpm/sf loading)
• Treated water transfer pumping	14.4 mgd	(one standby)

The capacities of system components relevant to the raw water storage reservoirs and the filters are discussed below.

RAW WATER STORAGE RESERVOIRS

Description of Existing Facilities

Raw water is pumped from the main or alternate raw water supply lines into the storage reservoirs and then flows by gravity to the clarifiers. Present operation is to pump to the south basin (No. 2) and feed the plant from the north basin (No. 1). The basins are used in series to maximize the hydraulic detention time to achieve initial sedimentation in the basins. The south basin also receives settled backwash water pumped from the backwash sump.

The elevation of the overflow structure for the raw water basins was originally 80.5 ft (assumed datum) but was lowered to 79.0 ft to comply with the requirements of the Department of Water Resources, Division of Safety of Dams. Water level in the basins is presently maintained at between 6 to 12 inches below the overflow level, or between elevations 78.0 to 78.5 ft.

The water surface level in the clarifiers is 76 ft, with approximately 1.5 ft of head loss in the clarifier feed piping from the basins at the clarifier design flow rate of 8.5 mgd each. Flow control valves on the clarifier feed lines from the basins control flow rate out of the basins based on water level in the clarifiers. Under normal

operating conditions, raw water flows by gravity from the basins to the clarifiers. In order to utilize the storage capacity of the basins below approximately 77.5 ft, it is necessary to use the raw water pump station to pump from the reservoirs to the clarifier feed lines.

The average bottom elevation of the basins is approximately 64 ft and the top of basins is approximately 82 ft. The basins each have interior dimensions of 440 ft wide by 580 ft long at the bottom. The interior levee slope is 2 horizontal to 1 vertical, and is approximately 36 ft long and 18 ft high. Based on a water surface elevation of 78 ft for normal operation, the total storage capacity of the basins is 60 MG. The bottom of the basins and the interior sides of the levee are asphalt-concrete paved.

Capacity Requirements

The State Department of Health Services (DHS) requires the City to maintain sufficient raw water storage to sustain the Date Canal anticipated down time (Reference 9). The Date Canal is scheduled to be cleaned once each month; however, cleaning has historically occurred about seven times per year. Each time the Date Canal is cleaned, the canal down time is a total of four days. As described above, during cleaning of the Date Canal, the City receives raw water from the Dahlia Lateral.

During cleaning or maintenance shutdown of the All American Canal, which supplies both the Date Canal and the Dahlia Lateral, there is no raw water supply available from either canal. It is reported that this situation has occurred once in the past 15 years for a period of four days (Reference 10). Emergency shut-down of the All American Canal, such as due to earthquake damage, would also result in no raw water supply from the canals. In that event, the only source of raw water would be from the plant raw water storage basins.

Capacity Evaluation

Raw water storage capacity requirements are evaluated below based on the average daily demand in the maximum month. As discussed in Section 3, the current average daily demand in the maximum month is 10.6 mgd, and is projected to increase at the projected population annual growth rate of 3.15 percent. A site exists west of the existing basins to add two basins, each of the same size as the existing basins. The raw water storage capacity of two, three, and four basins in terms of days of storage at the average daily demand during the maximum demand month is presented in Table 4.1.

The projected years in which additional raw water storage capacity will be required to meet various durations of supply interruption during the maximum demand month are shown in Table 4.2.

Table 4.1

Raw Water Storage Capacities

Year	Projected Max. Month Demand (mgd)	Storage Capacity[a] (days)		
		2 Basins[b]	3 Basins	4 Basins
1994	10.6	5.7	8.5	11.3
1995	10.9	5.5	8.3	11.0
1996	11.3	5.3	8.0	10.6
1997	11.6	5.2	7.8	10.3
1998	12.0	5.0	7.5	10.0
1999	12.4	4.8	7.3	9.7
2000	12.8	4.7	7.0	9.4
2001	13.2	4.5	6.8	9.1
2002	13.6	4.4	6.6	8.8
2003	14.0	4.3	6.4	8.6
2004	14.5	4.1	6.2	8.3
2005	14.9	4.0	6.0	8.1
2006	15.4	3.9	5.8	7.8
2007	15.9	3.8	5.7	7.5
2008	16.4	3.7	5.5	7.3
2009	16.9	3.6	5.3	7.1
2010	17.4	3.4	5.2	6.9
2011	18.0	3.3	5.0	6.7
2012	18.5	3.2	4.9	6.5
2013	19.1	3.1	4.7	6.3
2014	19.7	3.0	4.6	6.1
2015	20.3	3.0	4.4	5.9
2016	21.0	2.9	4.3	5.7
2017	21.6	2.8	4.2	5.6
2018	22.3	2.7	4.0	5.4
2019	23.0	2.6	3.9	5.2
2020	23.7	2.5	3.8	5.1
2021	24.5	2.4	3.7	4.9

[a] Based on basin storage volume of 60 MG each.

[b] Existing

Table 4.2

Raw Water Storage Capacity Projections

No. of Basins	Storage Volume (MG)	Outage Duration (days)	Available[a] Supply (mgd)	Year[b] Exceeded
2 (existing)	60 (existing)	4	15.0	2005
		5	12.0	1998
		6	10.0	1994
3	90 [c]	5	18.0	2011
		6	15.0	2005
		7	12.9	2001
		8	11.3	1996
		9	10.0	1994
4	120 [c]	6	20.0	2015
		7	17.1	2010
		8	15.0	2005
		9	13.3	2002
		10	12.0	1998
		11	10.9	1995

[a] At average day demand in maximum demand month.
[b] Year maximum month average day demand exceeds available supply from storage.
[c] Assuming same volume as each existing.

Recommendations

The timing of the need for additional storage capacity depends on the criteria established for duration of supply outage and demand during the outage. Water demand during an emergency supply could be limited by public information and reduction of distribution system pressure. However, the premise for evaluation of the raw water storage capacity is to maintain the normal level of service through an outage of reasonable duration. Therefore, the average demand in the maximum month is considered an appropriate criteria for this evaluation. Considering that water supply from the All-American Canal is crucial to the region and would be restored as soon as possible, it is assumed that the duration of an All-American Canal outage would be limited to five to seven days.

From the above criteria, additional storage capacity would be required immediately for seven days of storage or by the year 1998 for five days of storage. If one additional 30 MG reservoir is provided at these times, the second 30 MG reservoir would be required by the year 2001 for the seven day requirement and by

the year 2011 for the five day requirement. For the seven day requirement, additional storage beyond the 120 MG provided by four basins would be required by the year 2010. The 120 MG capacity of the four reservoirs would be adequate for ultimate conditions based on the five day storage requirement. It is recommended that the criteria of five days at the average day demand in the maximum demand month be established as the criteria for raw water storage. On that basis, an additional 30 MG basin will be required by the year 1998 and the second additional 30 MG basin will be required by the year 2011.

Cleaning

The basins have never been cleaned. Inspection of the north basin at low level has shown approximately 1 ft of sediment in the bottom of that basin. Since raw water flows through the south basin prior to the north basin, sediment levels in the south basin could be as high as several feet. Sediment in the basins would reduce storage capacity and should be cleaned out at some time in the future to maximize storage volume.

FILTRATION

Description of Existing Facilities

There are three existing filters, each 26 ft by 36 ft and having a surface area of 936 sf, for a total filter surface area of 2,808 sf. The filters are constructed of cast-in-place reinforced concrete. Backwashing of the filters is accomplished using a conventional pumped system. The filters were changed from single media to dual media in the early 1980's to improve performance and potentially increase capacity.

Capacity Requirements

The State Department of Health Services allows a maximum filter loading rate of 6 gpm/sf for dual filter media, gravity flow filters (Reference 11). The maximum loading rate for the City's filters has been previously rated at 5 gpm/sf, which corresponds to a total capacity of 20.2 mgd for all three filters and 13.5 mgd for two filters with one backwashing.

Filters with this type of media are generally operated at filtration rates of 2 to 6 gpm/sf. In this respect, the operating rate of 5 gpm/sf is at the high end of the typical range. However, these filters were designed hydraulically based on a lower loading rate for the single media design. A major effect of increasing the flow rate on the existing filters beyond the original design capacity would be to increase the head loss through the underdrain piping system, resulting in a rapid decrease in the total head available for the accumulation of suspended solids in the filter media. This increase in head loss could result in relatively short filter runs at the higher flow rates. The filter run time may be decreased to the point where the amount of backwash water became significant. In addition to decreasing filter run times, higher rates can sometimes cause effluent quality problems. High rates may cause solids to penetrate deeper into the filter bed resulting in a greater potential for solids breakthrough.

The maximum reported filtration rate experienced at the plant was 13.5 mgd, which corresponds to a filter loading rate of 3.3 gpm/sf. Performance of the filters at this loading rate was reported to be comparable to that at lower loading rates. Operators report that under present average loading conditions, each filter is backwashed once every three days. Under present maximum flows, run times per filter decrease somewhat to every two or three days. The total backwash cycle is approximately 25 minutes, approximately 15 minutes of which is actual backwashing. Each filter has individual flow control so with one filter under backwash the filtration rate on the other two filters does not increase. Due to the low frequency of backwashing, the decrease in total plant feed rate during backwashing has negligible effect on the total treatment capacity.

Capacity Evaluation

The capacity of the filtration system is evaluated based on maximum allowable filter loading rate at maximum day demand. Table 4.3 shows filter loading rates with one filter out of service for projected maximum day demand with three, four, and five filters.

Table 4.3

Evaluation of Filtration Capacity

Year	Projected Max. Day Demand (mgd)	Filter Loading Rate[a] (gpm/sf) 3 Filters[b]	4 Filters	5 Filters
1994	12.5	4.6	3.1	2.3
1999	14.6	5.4	3.6	2.7
2004	17.0	6.3	4.2	3.1
2009	19.9	6.3	4.9	3.7
2014	23.2	7.3	5.7	4.3

[a] Based on one filter backwashing and 936 sf per filter.
[b] Existing

Table 4.4 indicates the year in which additional filters would be required using a range of filter loading rates and projected maximum day demands. Based on a filter loading rate of 3.5 gpm/sf and the projected maximum day demand, additional filtration capacity will be required in 1998. Based on a filter loading rate of 5 gpm/sf, additional filtration capacity will not be required until the year 2010.

Table 4.4

Filtration Capacity Projections

No. of Filters	Surface Area (sf)	Loading Rate (gpm/sf)	Filtration[a,b] Capacity (mgd)	Year[c] Exceeded
3 (existing)	2808 (existing)	3.5	14.2	1998
		4.0	16.2	2003
		4.5	18.2	2006
		5.0	20.2	2010
		5.5	22.2	2013
4	3,744 [d]	3.5	18.9	2007
		4.0	21.2	2011
		4.5	24.3	2015
		5.0	27.0	2019
		5.5	29.7	-[e]
5	4,680 [d]	3.5	23.6	2015
		4.0	27.0	2019
		4.5	30.3	-[e]
		5.0	33.7	-[e]
		5.5	37.1	-[e]

[a] Based on 24 hour per day operation.
[b] Neglecting reduction in flow during backwash.
[c] Year projected maximum day demand exceeds filtration capacity.
[d] Assuming same area as each existing.
[e] Projected ultimate maximum day demand is 29 mgd.

It is assumed that new filters would be designed for 5 gpm/sf. On that basis and a 3.5 gpm/sf loading rate for the existing filters, a total of 2,038 sf of additional filter area would be required for the ultimate maximum day demand of 29 mgd. For a 5 gpm/sf existing loading capacity, an additional 1,200 sf of filter area would ultimately be provided. The existing filters are 936 sf. It is proposed that two additional filters be provided for the ultimate system. The second additional filter would be required in the year 2011 for the 3.5 gpm/sf existing loading capacity scenario and the year 2015 for the 5.0 gpm/sf existing loading capacity scenario.

Recommendations

For the purposes of evaluating existing filtration capacity, a filter loading rate capacity of 3.5 gpm/sf for the existing filters has been used based on past performance. It is recommended that the filters be field tested to determine their actual capacity rating. A detailed hydraulic analysis could also be performed to estimate the hydraulic capacity of the existing filters.

As discussed above, based on the 3.5 gpm/sf loading rate, an additional filter would be required by the year 1998. A second additional filter would be required by the year 2011 and would satisfy the ultimate filtration capacity requirement.

Spare Filter Requirement

Presently, with one filter out of service for maintenance or repair, the capacity of the remaining two filters at the 3.5 gpm/sf loading rate is 9.4 mgd. This is less than the maximum month average daily demand of 10.6 mgd, which occurs through the months of July and August. Two filters could not meet the daily demand through most of the summer months. If it was necessary to take one filter out of service for repair during the summer months, the treatment plant could not meet demand requirements. It is therefore recommended that an additional filter be provided as a spare for the existing filtration system.

Filter Controllers

It is understood that the existing filter controllers have an individual and combined capacity of 4.5 mgd and 13.5 mgd, respectively. If it is determined that the existing filters have capacity above 13.5 mgd, the controllers would have to be replaced when it becomes necessary to operate the filters at that rate.

Backwash Pumping

One 8,000 gpm pump backwashes the filters at a rate 8.5 gpm/sf. Considering the relatively long run times experienced by the filters, it is not anticipated that additional backwash pumping would be required for additional filters. However, the existing pump is original plant equipment and may need to be replaced in the near future. Impacts to the backwash pumping system associated with additional filters should be evaluated during design of the additional filters.

There is no standby backwash pump. A connection to the discharge header of the water distribution pumps is provided for supply of backwash water in the event the backwash pump is taken out of service for repair or maintenance. However, there is no means of positive control of the flow rate from the distribution pump station header. Since there is no flow control, and because of the large backwash flow rate required, use of the distribution pump station to supply backwash water significantly disrupts operation of the distribution system.

The State Department of Health Services (DHS), Drinking Water Division has determined that one duty backwash pump with the distribution system connection as a standby does not meet its requirements for reliability of the filtration system. The DHS has indicated that a standby backwash pump is necessary to ensure reliable operation of the filters. Addition of a standby backwash pump is therefore

recommended to meet DHS requirements. Remaining space in the existing pump room for a standby backwash pump is limited. Space for electrical gear for a standby pump would also be required in the electrical room. For the purposes of this Master Plan, it is assumed that the standby backwash pump would be located in the existing pump room to the west of the existing backwash pump and that available space in the electrical rooms is adequate for the additional electrical equipment associated with the pump.

Backwash Storage

Due to the long run times experienced by the filters, it is anticipated that the existing backwash sump has adequate volume to handle additional filters. It may be necessary to replace the pump which returns supernatant from the sump to the raw water basins with a higher capacity pump for additional filters. Impacts to these facilities associated with the addition of new filters would be evaluated during design.

Impacts to Treated Water Transfer Pumping

Filtered water is transferred to the treated water storage tanks by means of three vertical turbine pumps. Each pump is rated at 7.2 mgd for a total capacity of 21.6 mgd. With one pump as a standby, the pumping capacity is 14.4 mgd, which corresponds to the capacity of the filtration system of 14.2 mgd (3.5 gpm/sf filter loading rate). Addition of a 7.4 mgd duty filter by 1998 would increase the total filtration capacity to 21.6 mgd. If it is a requirement that one of the transfer pumps be available as a spare, it will be necessary to upgrade this pump station as part of the filter addition. For the purposes of this Master Plan, it is assumed that upgrading of the treated water transfer pump station will be required as part of each of the two proposed duty filter additions. Detailed requirements for additional treated water transfer pumping capacity should be evaluated during design of the additional filters.

Impacts to Clarifier Capacity Requirements

The existing clarifiers have a combined capacity of 17.0 mgd. Addition of duty filtration capacity in excess of 17.0 mgd will eventually necessitate an increase in clarification capacity. Addition of a 7.5 mgd duty filter by 1998 would increase the total filtration capacity to 21.7, which would require an increase in clarification capacity of 4.7 mgd. However, based on the flow projections shown in Table 4.3, this additional clarification capacity would not be needed until the year 2004.

As noted above, the projected ultimate maximum day demand is 29 mgd, which would require an additional 12 mgd of clarification capacity for ultimate conditions. This additional capacity would likely be provided by two 6 mgd clarifiers. The first additional clarifier would be required by 2004 and the second would be required by 2013.

SECTION 5

EVALUATION OF TREATED WATER STORAGE, TRANSMISSION, AND DISTRIBUTION SYSTEM

SECTION 5

EVALUATION OF TREATED WATER STORAGE, TRANSMISSION, AND DISTRIBUTION SYSTEM

This section analyzes the existing storage and distribution system for treated water and identifies improvements required to provide adequate capacity to serve existing and future development.

DESCRIPTION OF EXISTING DISTRIBUTION FACILITIES

The existing water distribution system is a single pressure zone, completely pumped system with off-line storage. Treated water storage and distribution facilities consist of the following:

- Three treated water storage reservoirs at the water treatment plant site having a total storage capacity of 10 MG.
- A 12,000 gpm main pump station located at the treatment plant which pumps treated water from storage into the distribution system.
- A remote storage reservoir of 5.0 MG capacity which receives treated water from the distribution system and stores it for domestic reserve and fire flow demand.
- A 3,500 gpm pump station at the remote storage reservoir site which pumps treated water from storage back into the distribution system.
- A transmission pipeline from the water treatment plant to the center of the existing service area.
- Other transmission and distribution pipelines.
- Two elevated storage tanks which are presently not in use.

Figure 5.1 shows the locations of the water treatment plant, the remote reservoir and pump station, and distribution pipelines 10-inch diameter in size and larger. The major components of the system are described below.

Treated Water Storage

All four of the treated water storage reservoirs presently in use are above-ground, circular, welded-steel tanks. Water is pumped from the tanks into the distribution system to maintain a pressure of approximately 60 psi. The water surface level in the tanks is independent of system pressure, except that the lower the water level, the lower the pump discharge rate into the system for a given system pressure.

Data on the treated water storage reservoirs are summarized in Table 5.1.

Figure 5.1 Existing Water Distribution System

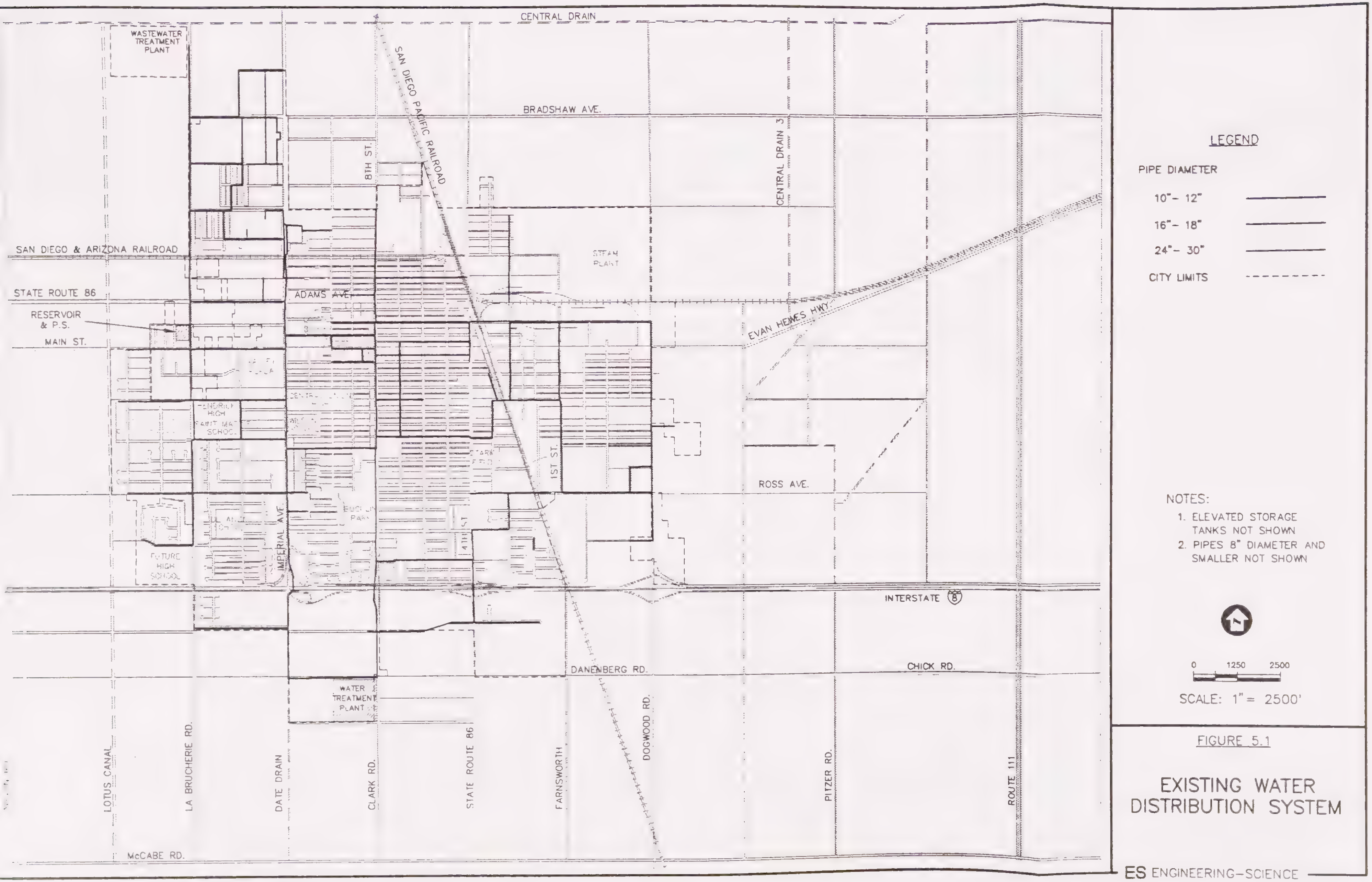

FIGURE 5.1

EXISTING WATER DISTRIBUTION SYSTEM

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Table 5.1

Treated Water Storage Reservoir Data

Reservoir	Type of Tank	Capacity (MG)	Diameter (ft)	Water Depth (ft)	Year Constructed
Treatment Plant					
Reservoir 1	Welded steel	2.5	133.5	22.5	1956
Reservoir 2	Welded steel	2.5	133.5	22.5	1956
Reservoir 3	Welded steel	5.0	194.0	22.5	1977
Remote Reservoir	Welded steel	5.0	150.0	38.0	1993

Water Treatment Plant Reservoirs

The three tanks at the treatment plant site are located along Clark Road across from Horne Street. Of the three reservoirs at the plant site, two have a storage capacity of 2.5 MG each and one has a capacity of 5.0 MG, for a total of 10.0 MG. The two 2.5 MG tanks are 133.5 ft in diameter with a maximum water depth of approximately 22.5 ft. The 5.0 MG tank has a diameter of 194 ft and a maximum water depth of approximately 22.5 ft.

The two 2.5 MG tanks were constructed in 1956 as part of the original plant construction. The upper portions of these tanks above the water line were reconstructed in 1980 and 1981. The 5.0 MG tank at the plant was added in 1977.

Remote Reservoir

The 5.0 MG remote tank is located on the west side of La Brucherie Road between Barbara Worth Avenue and Main Street. This reservoir was put into service in 1993. The tank diameter is 150 ft with a maximum water depth of 38 ft. The layout of the remote reservoir site includes space for a future 5.0 MG tank, and could accommodate a tank size up to 7.5 MG.

Elevated Storage Tanks

There are two elevated storage tanks in the system: one at 3rd Street and Commercial Avenue, with a capacity of 100,000 gallons, and one at 8th Street and Vine Street, with a capacity of 250,000 gallons. Both tanks rise 100 ft from base to overflow outlet. However, there is a ground elevation difference between the two tank sites of approximately 9 ft which results in the 3rd Street tank being lower than the 8th Street tank. This elevation difference limits the combined storage capacity

of the two tanks because the 3rd Street tank would overflow before the 8th Street tank could be filled. The actual combined storage capacity has been estimated to be 257,000 gallons (Reference 1).

Conversion of the distribution system to a 60 psi completely pumped system has made these tanks obsolete. Previously, the system floated off the 3rd Street tank level at approximately 40 psi. The elevated tanks were primarily for hydraulic control and provided minimal storage capacity. Since these tanks are no longer in service, they have not been evaluated as part of the existing distribution system.

Pump Stations

The two pump stations pump water from storage into the distribution system. The discharge rate of the pumps varies in response to demand. All of the duty pumps are horizontal, split case type with variable frequency drives.

Data on the treated water pump stations are summarized in Table 5.2

Table 5.2

Treated Water Distribution Pump Station Data

Pump Station	Number of Pumps	Individual Motor Rating (hp)	Motor Type	Individual Pump Capacity (gpm)	Pump Station Capacity
Treatment Plant	3	200	Variable	4,000 [a]	12,000 [b]
	1 [c]	100	Constant	3,000 [d]	
Remote	2	150	Variable	3,500 [a]	3,500

[a] At 60 psi
[b] Does not include full standby capacity.
[c] The constant speed pump is always a standby.
[d] At 45 psi

Treatment Plant

The distribution system pump station at the water treatment plant has three variable speed pumps and one constant speed pump. The three variable speed pumps were installed as part of the recently completed improvement project. The constant speed pump is always used as the standby, with the variable speed pumps available as duty pumps. The pumps are housed in the pump room of the plant operations building. The standby generator at the treatment plant has capacity to operate some but not all of the main pump station pumps. It is understood that the City is planning to replace the generator with a larger capacity system.

The capacity of the standby constant speed pump is limited because it was originally designed to operate with the 40 psi distribution system pressure. The rated capacity of the constant speed pump at 45 psi is 3,000 gpm; at 60 psi, its capacity is significantly less. If the standby pump cannot provide adequate pumping capacity at 60 psi in the event of a duty pump failure, the system pressure can be reduced temporarily which would increase the pumping capacity of all of the pumps. Also, the remote pump station presently has the capacity to provide short-term standby pumping in the event of a duty pump failure.

Remote Reservoir

The remote pump station is located at the remote reservoir site on La Brucherie Road south of Barbara Worth Avenue. The remote pump station has two variable speed pumps, one duty and one standby, and includes space and provisions for an additional duty pump. The remote pump station has adequate emergency power for the pump station with two duty pumps in operation.

Transmission and Distribution Pipelines

A 30-inch and 24-inch diameter transmission pipeline extends from the treatment plant to near the center of the existing service area. The pipeline is aligned along Clark Road across Interstate 8, and continues in 8th Street to Hamilton Avenue. A 24-inch and 18-inch transmission line extends west from the plant along Danenburg Road then turns north in Imperial Avenue and crosses the freeway. The distribution network is a combination of grid and branching system. Primary distribution pipelines are 10- to 18-inch diameter. Figure 5.1 shows the location of the transmission and loop and branch distribution pipelines.

Distribution System Operation

As previously noted, the system is presently operated as a completely pumped system; there is no on-line storage. The variable speed pumps are operated to match demand, with the number and speed of the pumps controlled based on maintaining system pressure at approximately 60 psi. System pressure is measured at the elevated tank site at 3rd and Commercial. The two pump stations can either be operated one at a time or together. If the three duty pumps at the main pump station operating at full speed cannot meet demand, the duty pump at the remote pump station is started. During low demand periods, the water treatment plant pump station refills the remote storage tank through the distribution system.

Prior to the recent completion of the improvement project, constant speed pumps at the treatment plant pump station were controlled on an on-off basis as a function of water level in the 3rd Street elevated tank. The capability still exists to operate the system using the elevated storage tanks with pump control based on 3rd Street tank level. However, system pressure using the elevated tanks is limited by the height of the tanks to around 40 psi. Considering the pressure limitation and that the elevated storage tanks provide only 0.26 MG of storage, it is anticipated that the system will be operated in the 60 psi mode on a permanent basis.

WATER DISTRIBUTION FACILITIES DESIGN CRITERIA

Evaluation of the treated water storage, transmission, and distribution system is based on the current and projected water demands developed in Section 3 (including fire flow). The adequacy of existing facilities and the need for new facilities are assessed. Table 5.3 lists the recommended design criteria for use in evaluation of the existing treated water storage and distribution facilities. These criteria will also be used for development and sizing of new facilities to serve future development.

EVALUATION OF TREATED WATER STORAGE CAPACITY

The total treated water storage requirement for the City is based on one day of average summer demand plus fire flow (References 1, 2). From Section 3, the present average daily demand during the maximum summer month is 10.6 mgd, increasing in the future at 3.15 percent annually. Also from Section 3, the maximum fire flow is 4,000 gpm for a four hour duration, which represents a volume of approximately 1.0 MG. Therefore, the present treated water storage requirement is 11.6 MG. As described above, existing treated water storage capacity is 15.0 MG.

Projected treated water storage volume requirements are listed in Table 5.4. Based on the 3.15 percent growth rate, construction of a second 5.0 MG storage tank at the La Brucherie Road remote site will be required by the year 2003. Ultimately, an additional 5.5 MG of storage will be required in addition to the existing 15.0 MG and the planned additional 5.0 MG remote tank. This additional storage is not projected to be required until the year 2013. It could either be provided at the treatment plant site or at a second remote site.

EVALUATION OF PUMPING CAPACITY

Duty Pumping Capacity

The criteria for pumping capacity has been established in previous planning studies (References 1, 2) as peak hour demand or maximum day demand plus fire flow. As discussed in Section 3, the estimated current peak hour demand is 13,500 gpm. The current estimated maximum day demand of 12.5 mgd, or 8,700 gpm, plus the established maximum fire flow of 4,000 gpm, is a total of 12,700 gpm. Peak hour and maximum day demand are projected to increase at a rate of 3.15 percent a year. Therefore, based on the above criteria, peak hour demand will determine pumping requirements for the combined pumping capacity of the treatment plant pump station and the remote pump station.

Although the combined capacity of the two pump stations can be used to meet peak hour demand, only the treatment plant pump station capacity is used to supply the daily demand. To meet the criteria of recharging the system within one day, the plant pump station must have adequate capacity to meet the maximum day demand and replenish fire flow storage. The present minimum capacity requirement for the treatment plant pump station is therefore 9,400 gpm, which is the maximum day demand of 8,700 gpm plus 700 gpm fire flow replenishment (4,000 gpm for four hours over 24 hours).

Table 5.3

Water Supply System Design Criteria

Item	Criteria
Water Mains:	
Maximum Allowable Velocity	
Peak Hour	10 ft/s
Peak Day plus Fire Flow	15 ft/s
Maximum Allowable Headloss	10 ft per 1000 ft of pipe
Hazen-Williams Roughness Coefficient	100
System Pressure: (at development pad elevation)	
Maximum Desired Pressure	60 psig
Maximum Allowable Pressure	80 psig
Minimum Pressure (Peak Day plus Fire Flow)	20 psig during Fire Flow
Minimum Pressure (Peak Hour Flow)	35 psig
Pump Stations:	
Minimum capacity	Supply peak hour demand, supply max. day demand plus fire flow; recharge within 1 day.
Minimum Number of Pumps	2 (1 standby pump equal to the largest duty pump in the station)
Standby Power	Requires permanent installation for prime pump stations and portable generator capability at all other pump stations
Maximum Suction/Discharge Velocities	8 ft/s with low headloss valves and appurtenances
Storage:	
Minimum Capacity	1 day demand of max month plus worst case fire flow

Table 5.4

Projected Treated Water Storage Requirements

Year	Daily[a] Demand (MG)	Fire[b] Flow (MG)	Treated Water Storage Required	Available	Surplus
1994	10.6	1.0	11.6	15.0 [c]	3.4
1995	10.9	1.0	11.9	15.0	3.1
1996	11.3	1.0	12.3	15.0	2.7
1997	11.6	1.0	12.6	15.0	2.4
1998	12.0	1.0	13.0	15.0	2.0
1999	12.4	1.0	13.4	15.0	1.6
2000	12.8	1.0	13.8	15.6	1.2
2001	13.2	1.0	14.2	15.0	0.8
2002	13.6	1.0	14.6	15.0	0.4
2003	14.0	1.0	15.0	20.0 [d]	5.0
2004	14.5	1.0	15.5	20.0	4.5
2005	14.9	1.0	15.9	20.0	4.1
2006	15.4	1.0	16.4	20.0	3.6
2007	15.9	1.0	16.9	20.0	3.1
2008	16.4	1.0	17.4	20.0	2.6
2009	16.9	1.0	17.9	20.0	2.1
2010	17.4	1.0	18.4	20.0	1.6
2011	18.0	1.0	19.0	20.0	1.0
2012	18.5	1.0	19.5	20.0	0.5
2013	19.1	1.0	20.1	25.5 [e]	5.4
2014	19.7	1.0	20.7	25.5	4.8
2015	20.3	1.0	21.3	25.5	4.2
2016	21.0	1.0	22.0	25.5	3.5
2017	21.6	1.0	22.6	25.5	2.9
2018	22.3	1.0	23.2	25.5	2.3
2019	23.0	1.0	24.0	25.5	1.5
2020	23.7	1.0	24.7	25.5	0.8
2021[f]	24.5	1.0	25.5	25.5	0.0

[a] Average day of the maximum demand month; refer to Section 3.
[b] Based on fire flow of 4,000 gpm for 4 hours; refer to Section 3.
[c] Existing treated water storage volume.
[d] Existing treated water storage volume plus additional 5.0 MG tank at remote site.
[e] Required ultimate treated water storage; additional 5.5 MG storage required.
[f] Projected build-out year; refer to Section 2.

The current available combined pumping capacity for both pump stations is 15,500 gpm at 60 psi. Higher pumping rates are possible using the existing pumps with system pressures lower than 60 psi. Presently, if the maximum fire flow is required during the peak hour, the system pressure will drop to a level at which a pumping rate of 19,500 can be achieved. The system pressure pumping at 19,500 gpm would be around 40 psi.

For the purpose of evaluating combined distribution pumping capacity, peak hour demand at 60 psi system pressure is established as the minimum criteria. Table 5.5 presents projected distribution pumping requirements for the combined capacity of the two pump stations. For the treatment plant pump station only, maximum day demand plus fire flow replenishment is established as the minimum criteria. Table 5.6 presents projected pumping capacity requirements for the treatment plant pump station only.

From Table 5.5, it will be necessary to install the lag (third) pump at the remote pump station by the year 1999, which would provide two duty pumps and one standby pump at that station. Installation of a fourth duty pump at the treatment plant pump station is projected to be required by the year 2006. Installation of this fourth duty pump in the remaining location at the pump station would result in a total of five pumps, four duty and one standby.

Expansion of the two existing pump stations is projected to provide adequate capacity through the year 2012. From that time to ultimate development, a total of 9,000 gpm of additional capacity is projected to be required. At least 5,000 gpm of this capacity would have to be provided at the treatment plant to provide adequate capacity to satisfy maximum daily demand. The remaining 4,000 gpm of additional capacity could either be provided at the treatment plant site or at a second remote site.

The final 4,000 gpm of pumping capacity should be located with the additional 5.5 MG of treated water storage capacity required beyond the second 5.0 MG tank at the La Brucherie remote site. Future consideration should be given to providing the final 4,000 gpm of projected ultimate pumping capacity and the final required ultimate 5.5 MG of treated water storage at a remote location in the eastern region of the City. However, for the purposes of this Master Plan, it is assumed that this pumping and storage will be provided at the treatment plant site.

Water Treatment Plant Standby Pump Replacement

As discussed previously in this Section, the standby pump at the water treatment plant pump station is not able to provide adequate standby capacity at the 60 psi system operating pressure. With one of the 4,000 gpm pumps at the treatment plant out of service, the remaining duty pump capacity available from the plant and the remote stations is 11,500 gpm, which is 2,000 gpm less than the present peak hour demand of 13,500 gpm. An evaluation of the performance curve for the standby pump shows that it could not provide 2,000 gpm pumping capacity at 60 psi, and therefore could not provide adequate standby capacity to meet present peak hour demands without the system operating pressure being lowered.

Table 5.5

Projected Combined Distribution Pumping Requirements

Year	Peak Hourly[a] Demand (gpm)	Pumping Capacity Available (gpm)	Pumping Capacity Surplus (gpm)
1994	13,500	15,500 [b]	2,000
1995	13,900	15,500	1,600
1996	14,400	15,500	1,100
1997	14,800	15,500	700
1998	15,300	15,500	200
1999	15,800	19,000 [c]	3,200
2000	16,300	19,000	2,700
2001	16,800	19,000	2,200
2002	17,300	19,000	1,700
2003	17,800	23,000 [d]	5,200
2004	18,400	23,000	4,600
2005	19,000	23,000	4,000
2006	19,600	23,000	3,400
2007	20,200	23,000	2,800
2008	20,800	23,000	2,200
2009	21,500	23,000	1,500
2010	22,200	23,000	800
2011	22,900	23,000	100
2012	23,600	32,000 [e]	8,400
2013	24,300	32,000	7,700
2014	25,100	32,000	6,900
2015	25,900	32,000	6,100
2016	26,700	32,000	5,300
2017	27,600	32,000	4,400
2018	28,400	32,000	3,600
2019	29,300	32,000	2,700
2020	30,200	32,000	1,800
2021[f]	31,200	32,000	800

[a] Current peak hour demand of 13,500 gpm escalated at 3.15 percent annually; refer to Section 3.
[b] Existing capacity (three at 4,000 gpm at plant and one at 3,500 gpm at remote).
[c] Existing capacity plus second 3,500 gpm duty pump at remote.
[d] Existing plus second duty 3,500 gpm duty pump at remote and fourth 4,000 gpm duty pump at plant; additional plant pumping capacity required per Table 5.6.
[e] Total additional 9,000 gpm capacity required in addition to existing plus remote and plant pump station expansions for ultimate capacity; at least 5,000 gpm must be provided at treatment plant (refer to Table 5.6).
[f] Projected build-out year; refer to Section 2.

Table 5.6

Projected Pumping Requirements for Treatment Plant Pump Station

Year	Daily[a] Demand (gpm)	Pumping Capacity Available (gpm)	Pumping Capacity Surplus (gpm)
1994	9,400	12,000 [b]	2,700
1995	9,600	12,000	2,400
1996	9,900	12,000	2,100
1997	10,200	12,000	1,800
1998	10,500	12,000	1,500
1999	10,800	12,000	1,200
2000	11,100	12,000	900
2001	11,500	12,000	500
2002	11,800	12,000	200
2003	12,100	16,000 [c]	3,900
2004	12,500	16,000	3,500
2005	12,900	16,000	3,100
2006	13,300	16,000	2,700
2007	13,700	16,000	2,300
2008	14,100	16,000	1,900
2009	14,500	16,000	1,500
2010	14,900	16,000	1,100
2011	15,400	16,000	600
2012	15,800	16,000	200
2013	16,300	21,000 [d]	4,700
2014	16,800	21,000	4,200
2015	17,300	21,000	3,700
2016	17,800	21,000	3,200
2017	18,400	21,000	2,600
2018	18,900	21,000	2,100
2019	19,500	21,000	1,500
2020	20,100	21,000	900
2021[e]	20,700	21,000	300

[a] Current maximum day demand of 8,700 gpm escalated at 3.15 percent annually plus 700 gpm fire flow replenishment; refer to Section 3.
[b] Existing capacity of three pumps at 4,000 gpm.
[c] Existing capacity plus fourth 4,000 gpm duty pump.
[d] Total additional 5,000 gpm capacity required in addition to existing plus fourth 4,000 gpm pump for ultimate capacity.
[e] Projected build-out year; refer to Section 2.

Although the distribution system is designed to either operate as a pumped system up to 60 psi or to float off the elevated storage tanks at a pressure of approximately 40 psi, direct pumping to 60 psi is the primary operating mode. Lowering the distribution system operating pressure below 60 psi whenever a pump at the treatment plant is taken out of service in order to be assured of adequate pumping capacity would result in unacceptable operational requirements and reductions in the level of service. It is therefore recommended that the spare pump at the treatment plant be upgraded to match the performance of each of the duty pumps, or be replaced with a new pump identical to the existing duty pumps.

The existing standby pump is a 100 hp constant speed pump and would be replaced with a 200 hp variable speed pump. A spare variable frequency variable speed drive (VFD) and isolation transformer are provided for the existing variable speed pumps. Space is provided in the first floor electrical room for a future VFD for the constant speed pump, and in the second floor electrical room for a future isolation transformer. Under this arrangement, there would be five VFD's for the four pumps, with one VFD available as a spare. For the purposes of this Master Plan, it is assumed that an additional VFD and accompanying isolation transformer would be provided as part of upgrading of the standby pump. During design of the upgrade for the standby pump, the use of the spare VFD for the standby pump could be evaluated. As part of the upgrade of the standby pump, the control logic for the pumps would be modified to allow any of the pumps to be designated as the standby.

COMPUTER MODELING OF EXISTING WATER DISTRIBUTION SYSTEM

An evaluation of the existing water distribution system was performed using the Cybernet program developed by Haestad Methods. Cybernet provides for graphical interface with the hydraulic modeling software KYPIPE2 through AutoCAD. The CYBERNET program provides a graphical approach to the analysis of water distribution networks using visual imagery for data input and editing, reviewing results, creating color coded maps, creating pressure and flow contour mapping, and other functions.

Existing System Data

All known distribution pipelines were incorporated into the hydraulic model by digitizing information from water distribution system maps, as-built drawings, and topographical maps provided by the City. This information included size and length for all pipes, and ground surface elevations.

Reservoirs and Pump Stations

The treatment plant reservoirs and pump station and the remote reservoir and pump station were included in the model. Pump rates were based on actual pump curves and reservoirs were modelled using tank volume and water levels.

Demand Distribution

Demand was distributed throughout the system based on water meter readings for the month of August 1993. This month was selected as the current maximum demand month based on plant pumping records for the last year. Meter readings were summed by node service area and applied to the nodes as the average day demand in the maximum month. Demands other than average day in the maximum month were modelled based on the maximum month average day distribution multiplied by the ratio of the demand modelled over the maximum month average day demand.

Friction Factor

A uniform Hazen-Williams friction factor of 100 was used for all piping based on the results of the model calibration as described below. This friction factor was determined through calibration of the model with actual field data. A friction factor of 100 is within the lower range of typical friction factors. Since the friction factor in this case includes minor losses, it is considered a reasonable value. In reality, the friction factor will be lower in areas with older pipes. However, sufficient data were not available to differentiate friction factor by area in this case.

Minor Losses

Minor losses in the distribution system were not modelled. It was assumed that minor losses would be distributed evenly throughout the system and would be reflected in the selected friction factor. Minor losses at the pump stations were included in the model.

Model Calibration

For the purpose of calibrating the computer model with the actual distribution system, the City performed pressure tests at eight locations within the system. These tests were performed during both high and low flow demand conditions. Pumping rate, number of pumps in operation, and pump speed were recorded at the time of pressure measurement. The computer model was run using parameters based on the field recorded data to verify that the model results were consistent with field measurements. After running several calibration scenarios, it was determined that a fiction factor of 100 provided the best correlation between the model and the field data. As noted above, this friction factor value is consistent with expectations for this distribution system.

The calibration runs were repeated with the friction factor value of 100 and the model results correlated well with the field data. The correlation was best for the higher demand periods, and poorer for the low demand data. It is expected that the decline in correlation at low pumping rates is due to the limitations of the model in terms of its ability to simulate pump performance characteristics. Modelled pressures for the high flow conditions were generally within 1 to 3 psi of the field recorded pressures. Calibration data are provided in Appendix B.

Modeling Results

Following calibration of the distribution network model, various scenarios were modelled under present demand conditions. The following flow conditions were modelled based on the existing demands developed in Section 3:

- Annual average day demand of 10.4 mgd
- Maximum day demand of 12.5 mgd
- Peak hour demand of 13,500 gpm

For demands other than average day in the maximum month, the demand distribution was increased or decreased based on the modelled demand scenario proportional to the maximum month average day demand. Demand scenarios were modelled for: the treatment plant pump station with the remote pump station available if needed, the plant pump station only, and the remote pump station only. Figure 5.2 shows iso-pressure contours for the existing distribution system for maximum month average day demand with both pump stations in operation. Figure 5.3 shows iso-pressure contours for the existing distribution system for peak hour demand with both pump stations operating. Pressure contours are in psi.

Fire flow demands were modelled with simultaneous maximum day demand. Figure 5.4 shows iso-flow contours for available fire flow under maximum day demand conditions with a residual pressure of 35 psi. Fire flow contours are in gpm.

The results of the hydraulic modelling of the existing distribution system are provided in Appendix C.

RECOMMENDED IMPROVEMENTS TO EXISTING DISTRIBUTION SYSTEM

From a review of the existing system pressure contours shown on Figures 5.2 and 5.3, significant pressure drops occur for the peak hour condition at the following locations:

1. The south-west area of the City north of Interstate 8 and west of 8th Street
2. The area north of Ross Avenue in the vicinity of 4th Street
3. Along Ross Avenue at 1st Street

The south-west area of the City north of Interstate 8 is not directly served by a distribution main. Water must travel from the mains which cross Interstate 8 at Imperial Avenue and 8th Street. The south-west area is also the highest ground in the City. These two factors combine to result in system pressures less than 54 psi in the area. A direct supply across the freeway at La Brucherie Road is recommended as follows:

- Install a 2,400 ft, 27-inch (or 30-inch) pipeline in La Brucherie Road from the 18-inch dead-end line at Wake Avenue to the 18-inch line at Ocotillo Drive. This pipeline would be part of a transmission line which will ultimately convey water to the west side of the City. This connection pipeline is generally consistent with the 1982 Master Plan which recommended an 18-inch line in

La Brucherie from the 18-inch line along this alignment. In the short-term it will provide a more direct supply for the south-western portion of the city and in the long-term it will be part of a transmission main from the treatment plant to the west side of town.

- Install a 24-inch and 18-inch line parallel to the existing 18-inch line in Danenburg Road and along the Date Drain, from the treatment plant to the 18-inch in Wake Avenue. This is consistent with the recommendations of the 1982 Master Plan Update which recommended a parallel 18-inch. The pipeline in La Brucherie Road described above will increase the flow requirements for this section of pipeline. At Wake Avenue, the flow will split to the north and west through the existing 18-inch lines. The parallel segment in Danenburg Road is recommended to be 24-inch because it will be part of the ultimate transmission system.

The south-west area of the City also does not have a direct connection to the primary distribution lines. Water must flow north to the 18-inch line in Orange Avenue then south in the 12-inch in 1st Street. This situation can be alleviated by connecting the 12-inch line in Ross Avenue to the 12-inch line in 1st. This will require a 325 ft section of 12-inch in Ross to 1st Street.

In addition to the 12-inch connection in Ross Avenue described above, it is recommended that a 1,700 ft, 12-inch line in 4th Street from Ross to Hamilton Avenue be provided to improve looping in the south-east area and to replace existing 4-inch and 6-inch lines. This connection plus the line in Ross Avenue recommended above, would complete two 12-inch and 18-inch loops which would improve supply to the area north of Ross.

In summary, the recommended pipeline improvements for the existing system are as follows, in order of priority:

1. Southwest Area
   a. Install a 2,400 ft, 27-inch (or 30-inch) line in La Brucherie Road from Wake Avenue, across Interstate 8, to Ocotillo Drive.
   b. Install a 1,300 ft, 18-inch line parallel to the existing 18-inch along the extension of Imperial Avenue (Date Drain) from Danenburg Road to Wake Avenue.
   c. Install a 2,500 ft, 24-inch line parallel to the existing 18-inch in Danenburg from the treatment plant to the extension of Imperial Avenue.
2. Southeast Area. Install a 325 ft, 12-inch connection pipeline from the end of the 12-inch dead-end in Ross Avenue to the 12-inch at Ross and First Street.
3. East-Central Area. Install a 1,700 ft, 12-inch line in 4th Street from the 12-inch line at Ross Avenue to the 18-inch line in Hamilton Avenue.

Figure 5.5 shows the recommended pipeline improvements for the existing system. Figures 5.6 and 5.7 show pressure contours for the existing system with the recommended improvements, for maximum month average day demand and peak hour demand conditions. Figure 5.8 shows iso-flow contours for available fire flow

with the existing system improvements under current maximum day demand conditions and a residual fire flow pressure of 35 psi. Pressure contours are in psi and fire flow contours are in gpm. The results of the hydraulic modeling for the existing distribution system with the recommended improvements are included in Appendix C.

From Figures 5.6 and 5.7, the model shows that the recommended pipeline improvements reduce flow through the existing transmission pipelines and provide a more direct flow path to the north-west area of the City. This results in a decrease in head loss from the treatment plant pump station to the pressure control station at 3rd and Commercial. For a given pressure control setpoint of 60 psi, the discharge pressure at the treatment plant is decreased with the recommended pipeline improvements. This results in more efficient pumping, but does tend to lower the system pressure in the south and south-west areas of the City. This effect partially counteracts the benefits of improved distribution to these areas. The pipeline improvements are recommended because they will also improve available fire flow throughout the City and are necessary for the ultimate distribution system. If the pressures shown on Figures 5.6 and 5.7 in the south and south-west areas are not adequate with the recommended improvements, consideration should be given to increasing the control setpoint pressure once the improvements have been implemented.

PROPOSED ULTIMATE TRANSMISSION AND DISTRIBUTION PIPELINE SYSTEM

The ultimate transmission and distribution pipeline system is proposed to consist of a major transmission pipeline loop around the City, connecting to existing transmission pipelines at key locations. This transmission pipeline would supply loop pipelines and distribution pipelines in the future service area. Future pipelines have been sized for relatively low head losses of the same order of magnitude as the existing system. It is necessary to size the future pipes for these relatively low head losses to provide for uniform flow through the system and to maintain pumping heads consistent with the existing distribution pumping system.

Figure 5.9 shows the proposed ultimate transmission and distribution pipeline system. Figure 5.10 is a schematic of the ultimate system. Figure 5.11 shows pressure contours for the ultimate system based on the following:

- Projected ultimate average day demand in the maximum month of 24.4 mgd
- Treatment plant pump station capacity expanded to 25,000 gpm
- Remote pump station capacity expanded to 7,000 gpm.

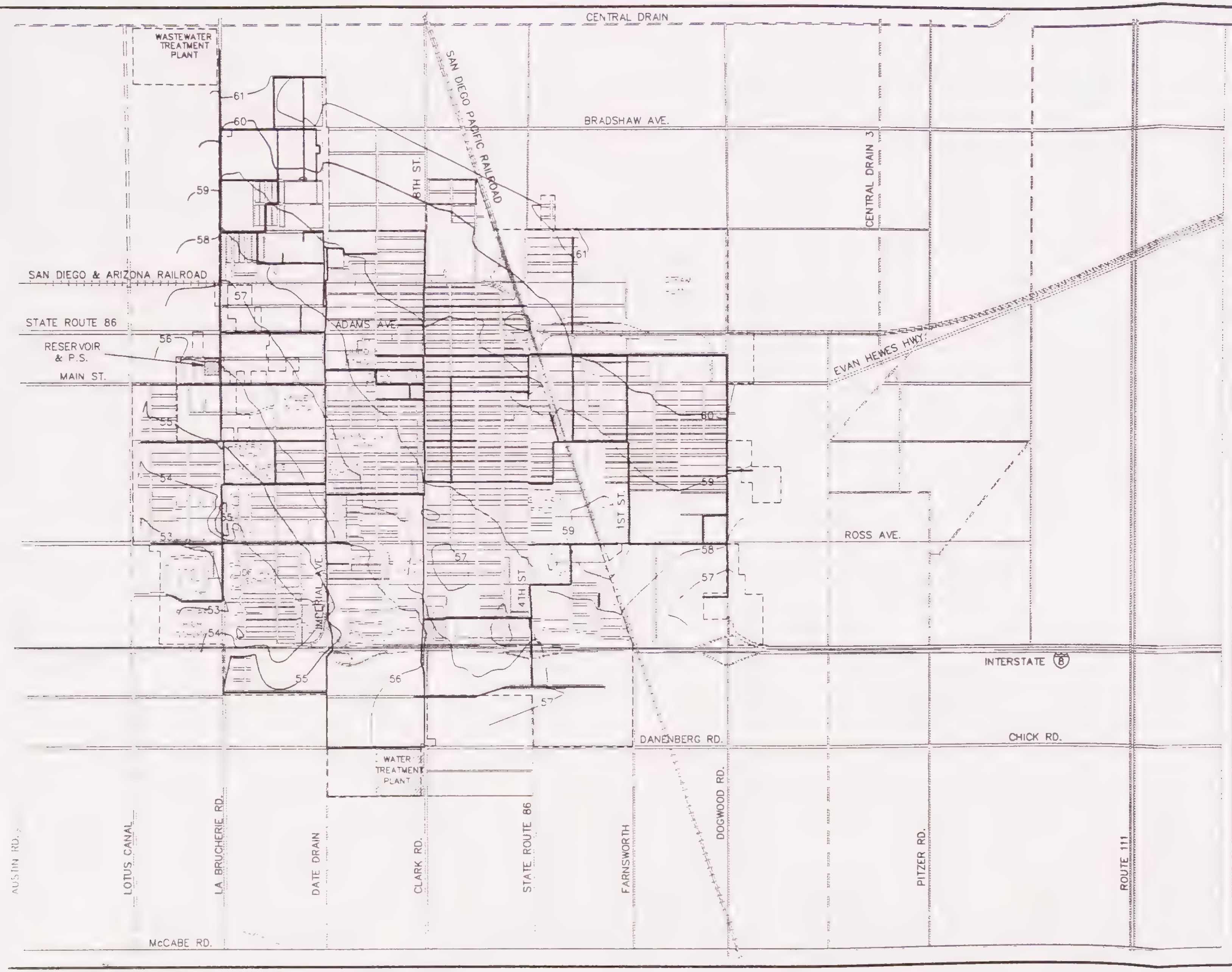
Figure 5.12 shows pressure contours for the ultimate system for the projected ultimated peak hour flow of 32,500 gpm and the pump station modifications noted for Figure 5.11. Figure 5.13 shows iso-flow contours for available fire flow with the ultimate system under maximum day demand conditions with a residual fire flow

pressure of 35 psi. Pressure contours are in psi and fire flow contours are in gpm. The results of the hydraulic modeling for the ultimate distribution system are provided in Appendix C.

PRESSURE MONITORING STATIONS

To assist in optimizing the operation of the system, it is desirable to install pressure sensors throughout the system. The sensors would transmit their signal back to the water treatment plant and be recorded. Key locations for these measurements would be at the higher elevations and known low pressure areas.

Figure 5.2 Existing Water Distribution System Pressure Contours - Average Day



NOTES:

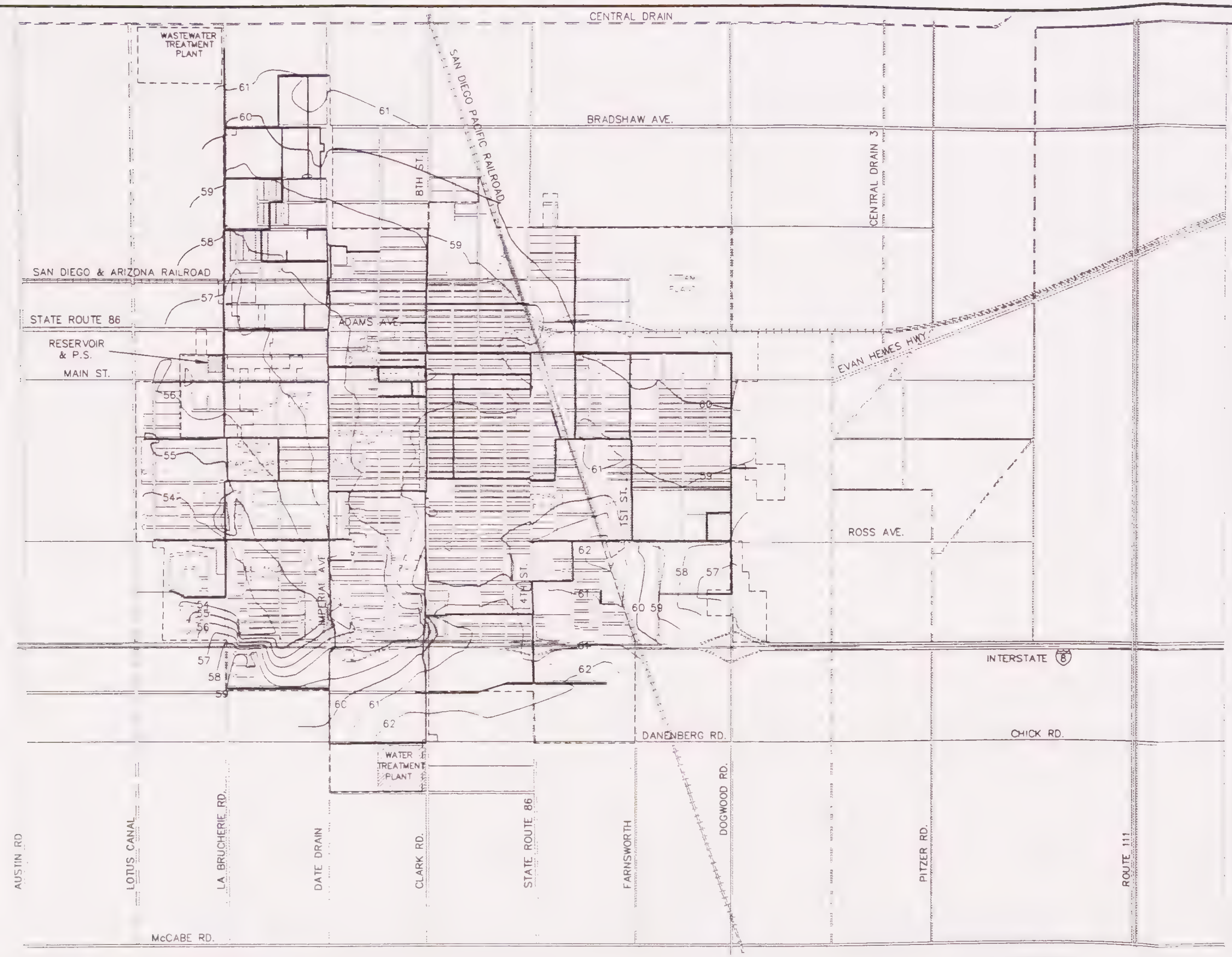
1. PRESSURE CONTOURS SHOWN ARE FOR AVERAGE DAY DEMAND IN THE MAXIMUM DEMAND MONTH: 10.6 MGD.
2. PRESSURE CONTOURS SHOWN ARE BASED ON TWO PUMPS OPERATING AT THE TREATMENT PLANT.

FIGURE 5.2

EXISTING WATER DISTRIBUTION SYSTEM PRESSURE CONTOURS AVERAGE DAY

ES ENGINEERING–SCIENCE

30, 1994 – 16:12:19 724452\ECFIG5-2.DWG CAD 5



LEGEND

PIPE DIAMETER

10"- 12"

16"- 18"

24"- 30"

CITY LIMITS

PRESSURE CONTOURS (PSI)

NOTES:

1. PRESSURE CONTOURS SHOWN ARE FOR PEAK HOUR DEMAND IN THE MAXIMUM MONTH: 13,500 GPM.
2. PRESSURE CONTOURS ARE BASED ON THREE PUMPS OPERATING AT THE TREATMENT PLANT AND ONE PUMP OPERATING AT THE REMOTE PUMP STATION.

0 1250 2500

SCALE: 1" = 2500'

FIGURE 5.3

EXISTING WATER DISTRIBUTION SYSTEM PRESSURE CONTOURS PEAK HOUR

ES ENGINEERING-SCIENCE

ep: 30, 1994 - 12:03:09 724452\ECFIG5-3.DWG CAD 5

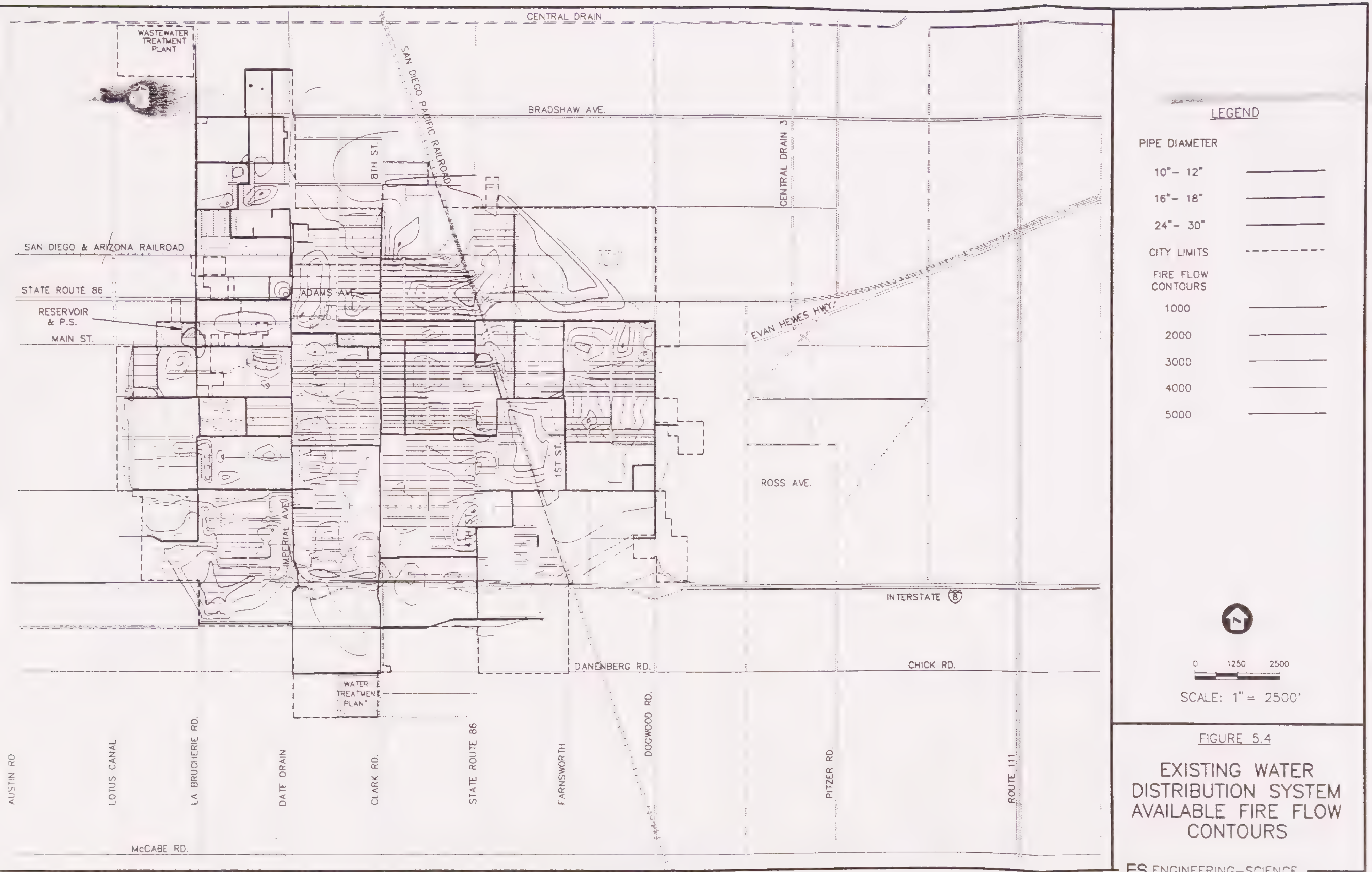


FIGURE 5.4

EXISTING WATER DISTRIBUTION SYSTEM AVAILABLE FIRE FLOW CONTOURS

ES ENGINEERING-SCIENCE

t 01, 1994 - 16:24:35 724452\ECFIG5-4.DWG CAD 5

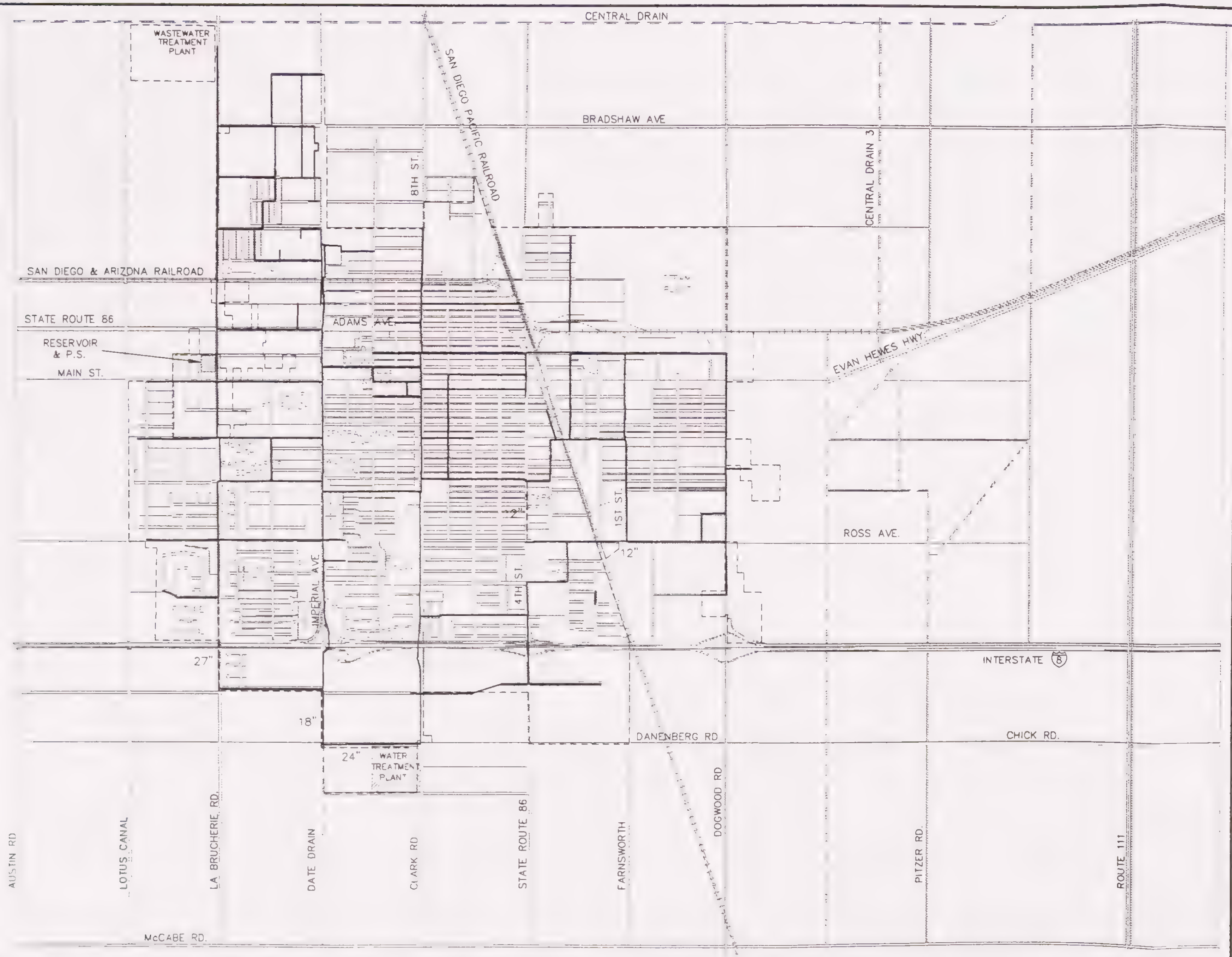


FIGURE 5.5

EXISTING WATER DISTRIBUTION SYSTEM WITH IMPROVEMENTS

ES ENGINEERING-SCIENCE

...t 30, 1994 - 12:04.52 724452\ECFIG5-5.DWG CAD 5

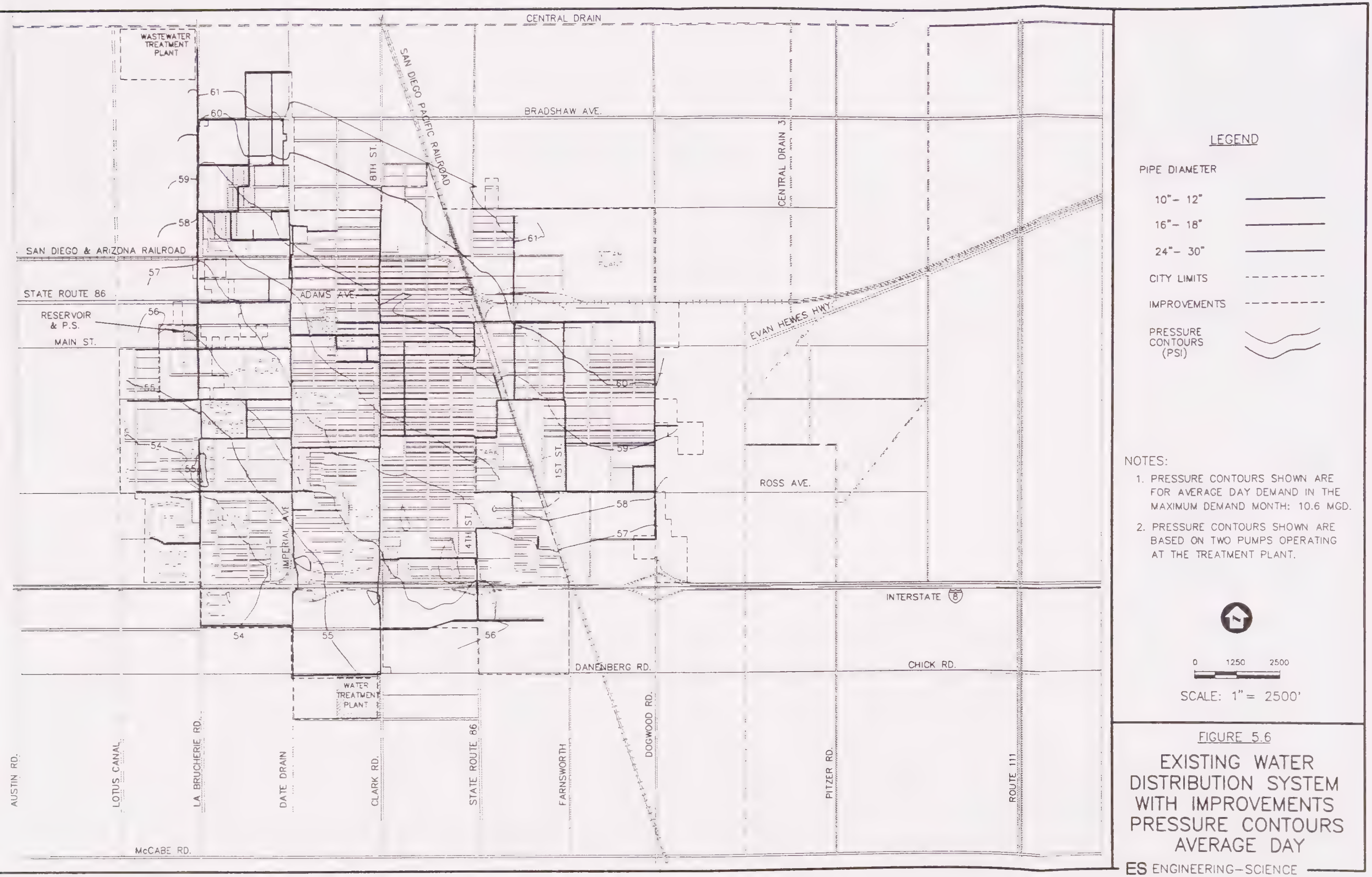
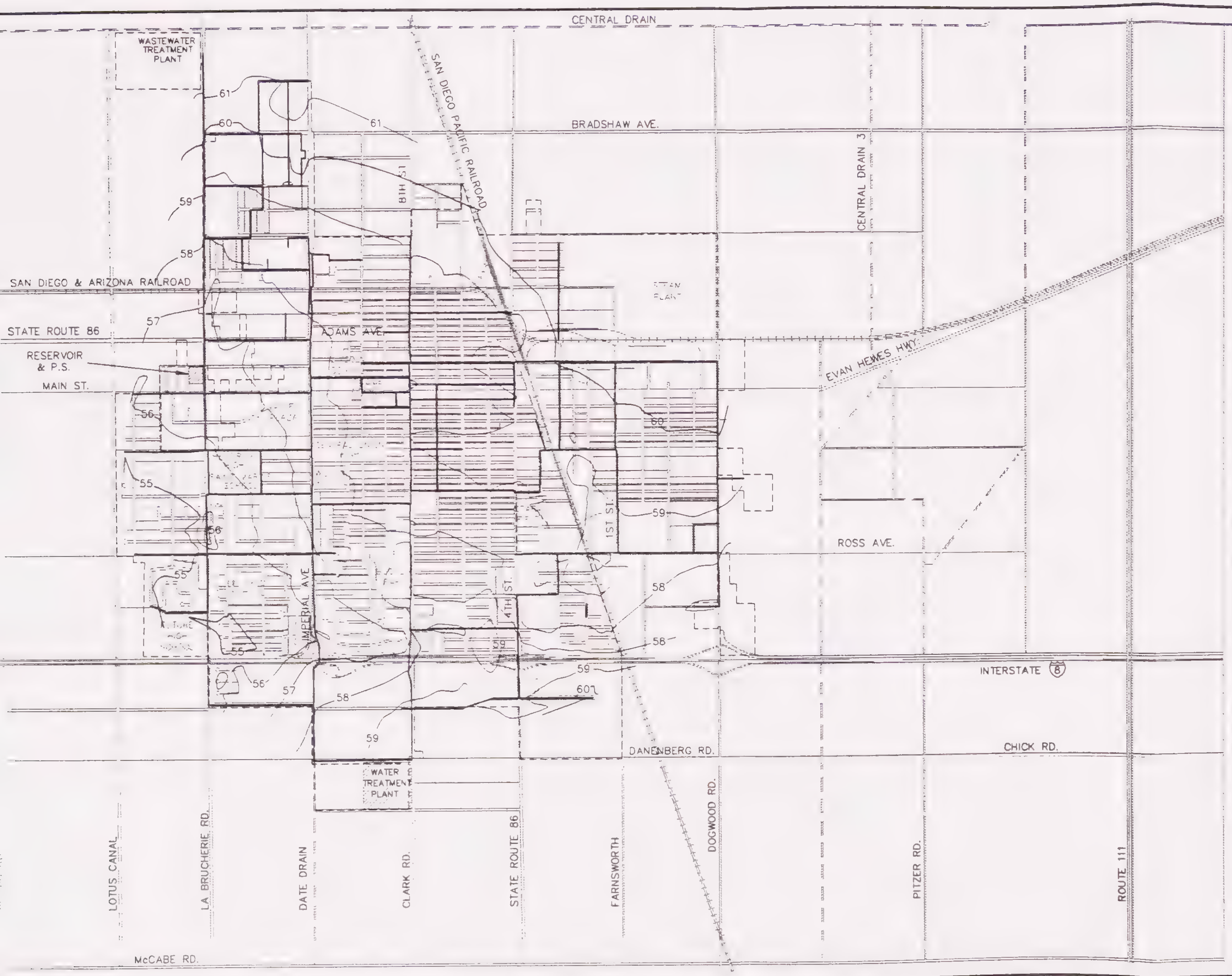


FIGURE 5.6
EXISTING WATER DISTRIBUTION SYSTEM WITH IMPROVEMENTS PRESSURE CONTOURS AVERAGE DAY

pt 29, 1994 - 18:20:18 724452\ECFIG5-6.DWG CAD 5



LEGEND

PIPE DIAMETER

10"– 12"

16"– 18"

24"– 30"

CITY LIMITS

IMPROVEMENTS

PRESSURE CONTOURS (PSI)

NOTES:

1. PRESSURE CONTOURS SHOWN ARE FOR PEAK HOUR DEMAND IN THE MAXIMUM MONTH: 13,500 GPM.
2. PRESSURE CONTOURS ARE BASED ON THREE PUMPS OPERATING AT THE TREATMENT PLANT AND ONE PUMP OPERATING AT THE REMOTE PUMP STATION.

0 1250 2500

SCALE: 1" = 2500'

FIGURE 5.7

EXISTING WATER DISTRIBUTION SYSTEM WITH IMPROVEMENTS PRESSURE CONTOURS PEAK HOUR

ES ENGINEERING-SCIENCE

29, 1994 – 18:44:42 724452\ECFIG5-7.DWG CAD 5

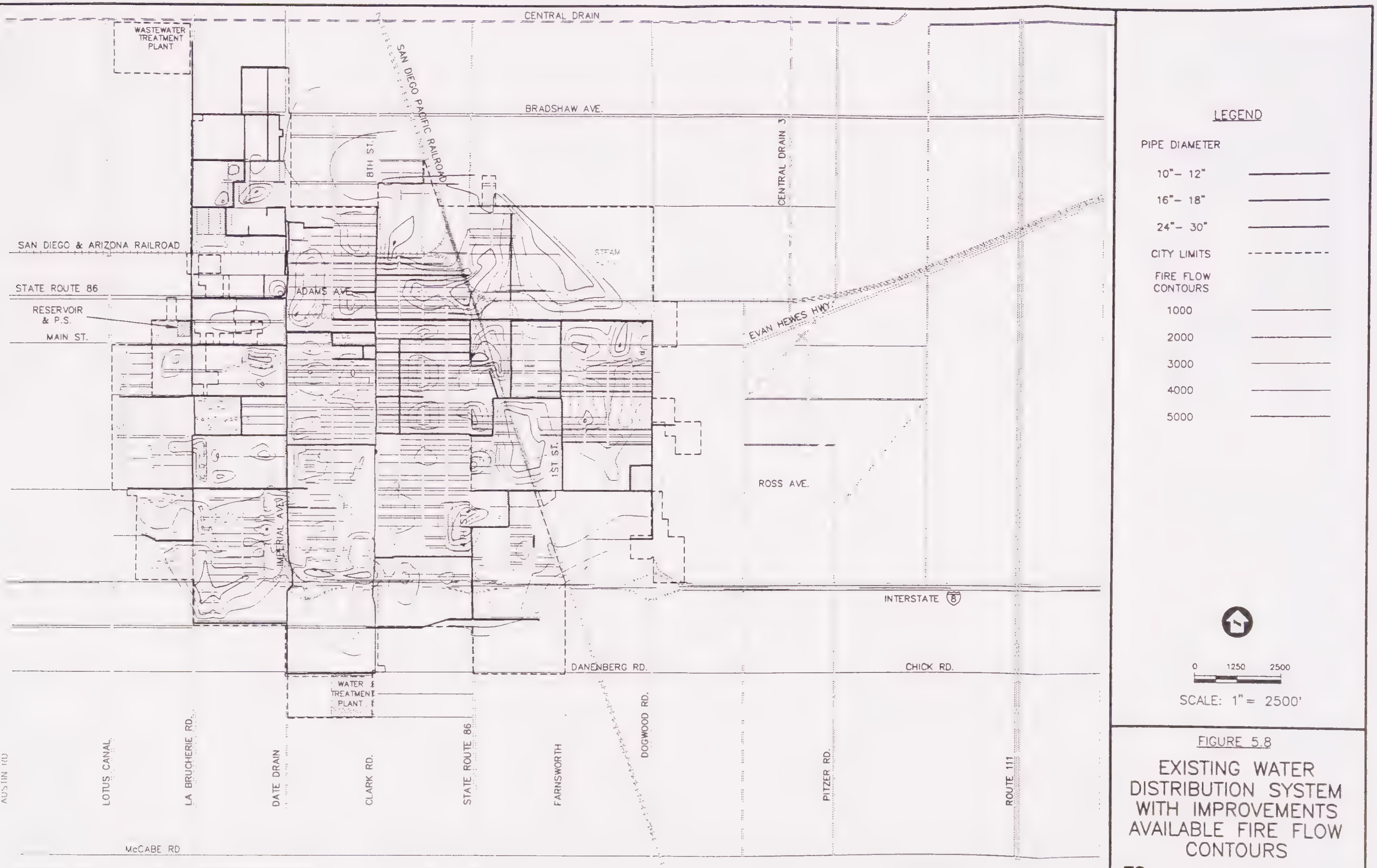


FIGURE 5.8

EXISTING WATER DISTRIBUTION SYSTEM WITH IMPROVEMENTS AVAILABLE FIRE FLOW CONTOURS

ES ENGINEERING-SCIENCE

01, 1994 – 18:10:53 724452\ECFIG5-8.DWG CAD 5

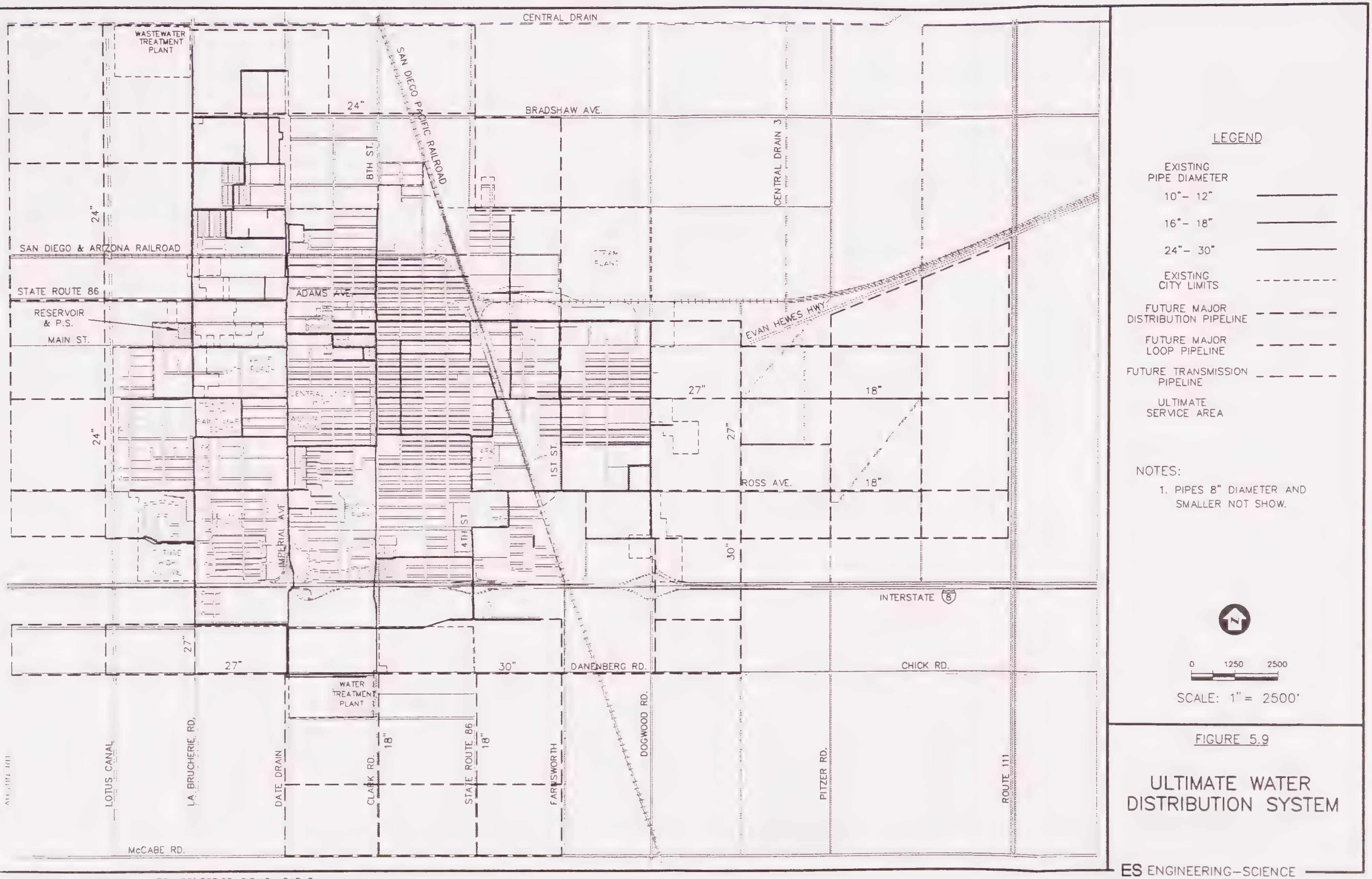


FIGURE 5.9

ULTIMATE WATER DISTRIBUTION SYSTEM

30, 1994 – 12:10:10 724452\ECFIG5-9.DWG CAD 5

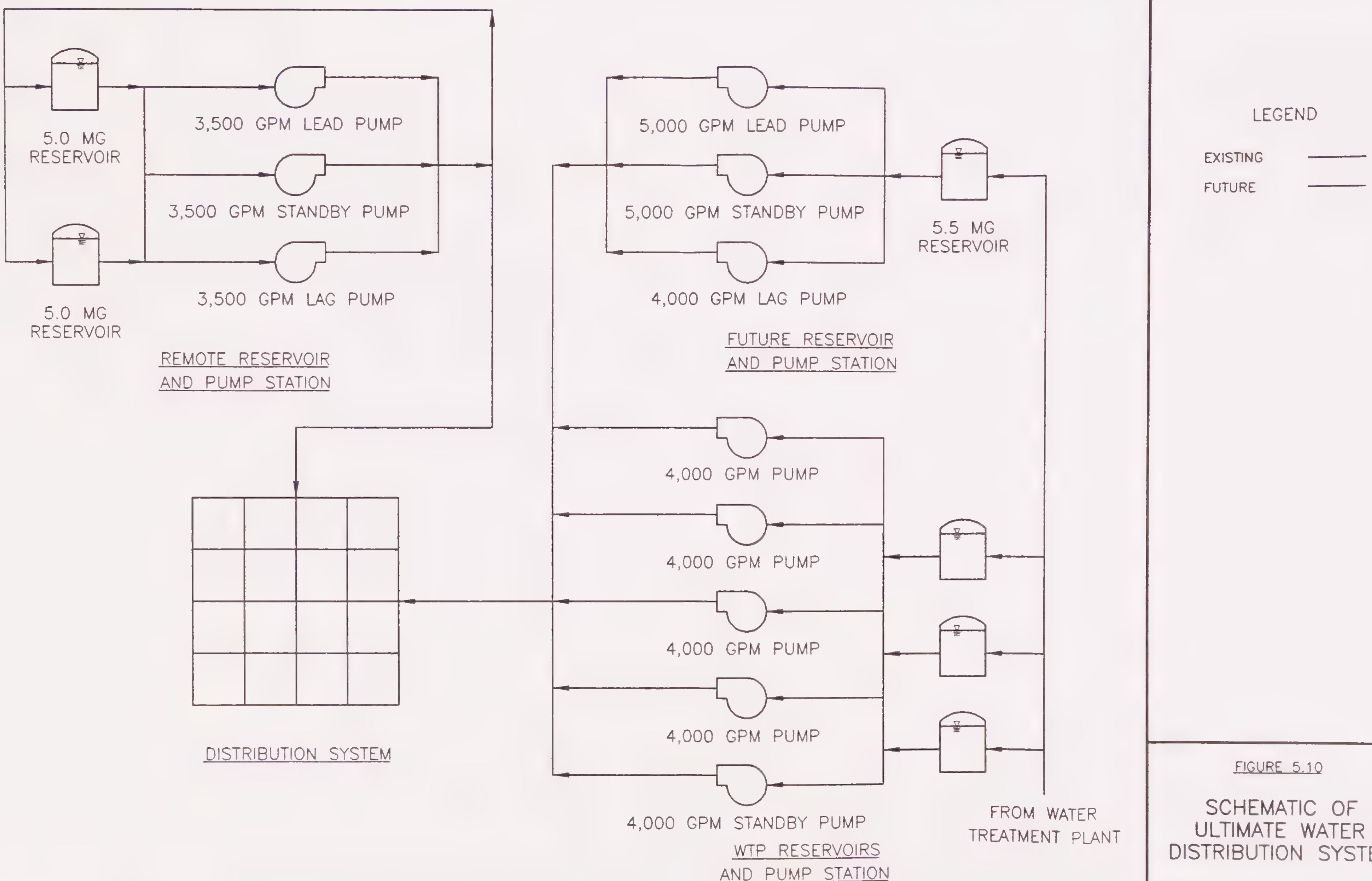
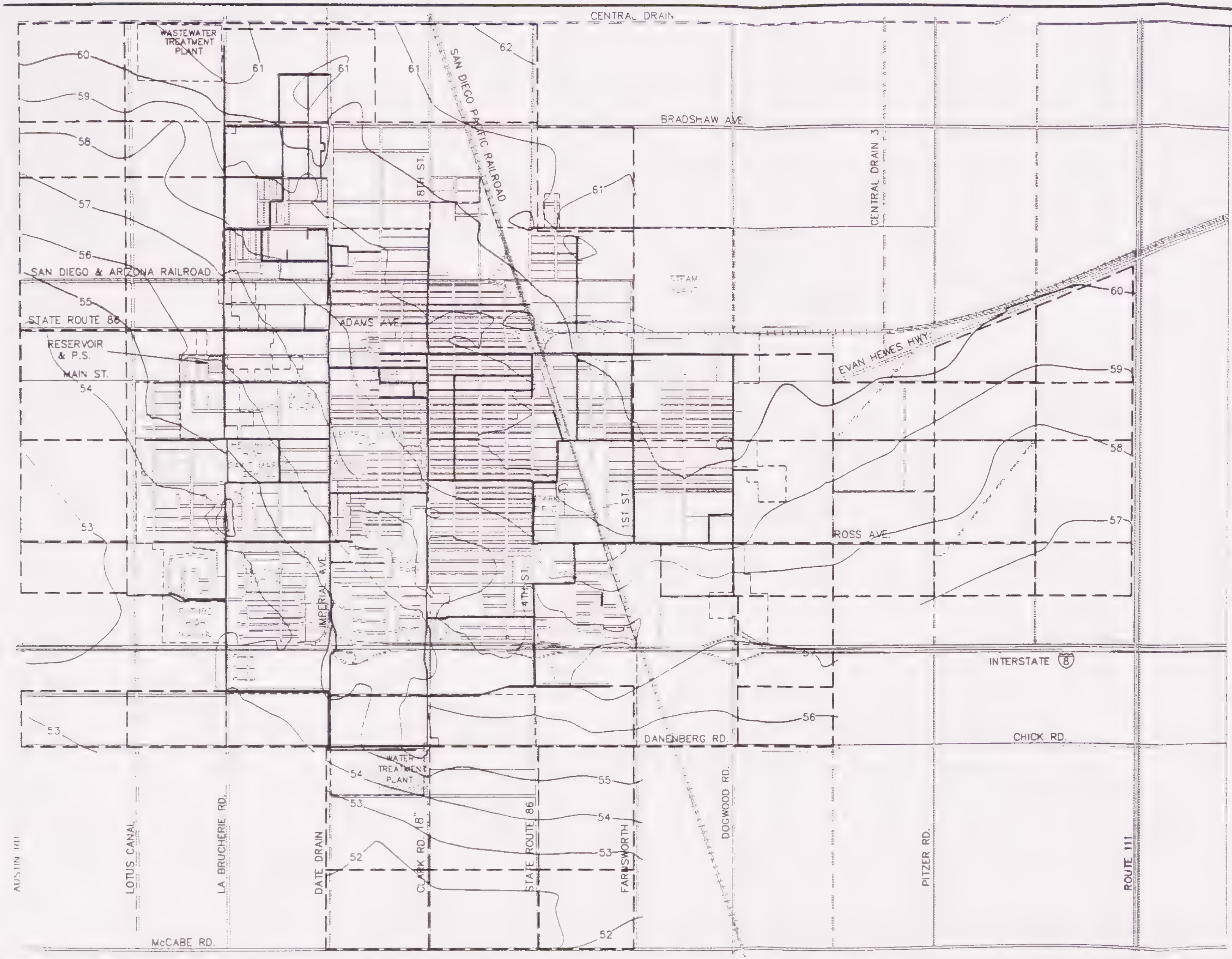


FIGURE 5.10

SCHEMATIC OF ULTIMATE WATER DISTRIBUTION SYSTEM

ES ENGINEERING-SCIENCE

02, 1994 – 18:40:20 ACAD\724452\ECFIG510.DWG CAD 5



LEGEND

EXISTING PIPE DIAMETER
10" – 12"
16" – 18"
24" – 30"

EXISTING CITY LIMITS

FUTURE MAJOR DISTRIBUTION PIPELINE

FUTURE MAJOR LOOP PIPELINE

FUTURE TRANSMISSION PIPELINE

ULTIMATE SERVICE AREA

PRESSURE CONTOURS (PSI)

NOTES:

1. PRESSURE CONTOURS SHOWN ARE FOR AVERAGE DAY DEMAND IN THE MAXIMUM DEMAND MONTH: 24.4 MGD.
2. PRESSURE CONTOURS ARE BASED ON FOUR PUMPS OPERATING AT THE TREATMENT PLANT AND ONE PUMP OPERATING AT THE REMOTE PUMP STATION.

0 1250 2500

SCALE: 1" = 2500'

FIGURE 5.11

ULTIMATE WATER DISTRIBUTION SYSTEM PRESSURE CONTOURS AVERAGE DAY

ES ENGINEERING–SCIENCE

: 30, 1994 – 12:12:09 724452\ECFIG511.DWG CAD 5

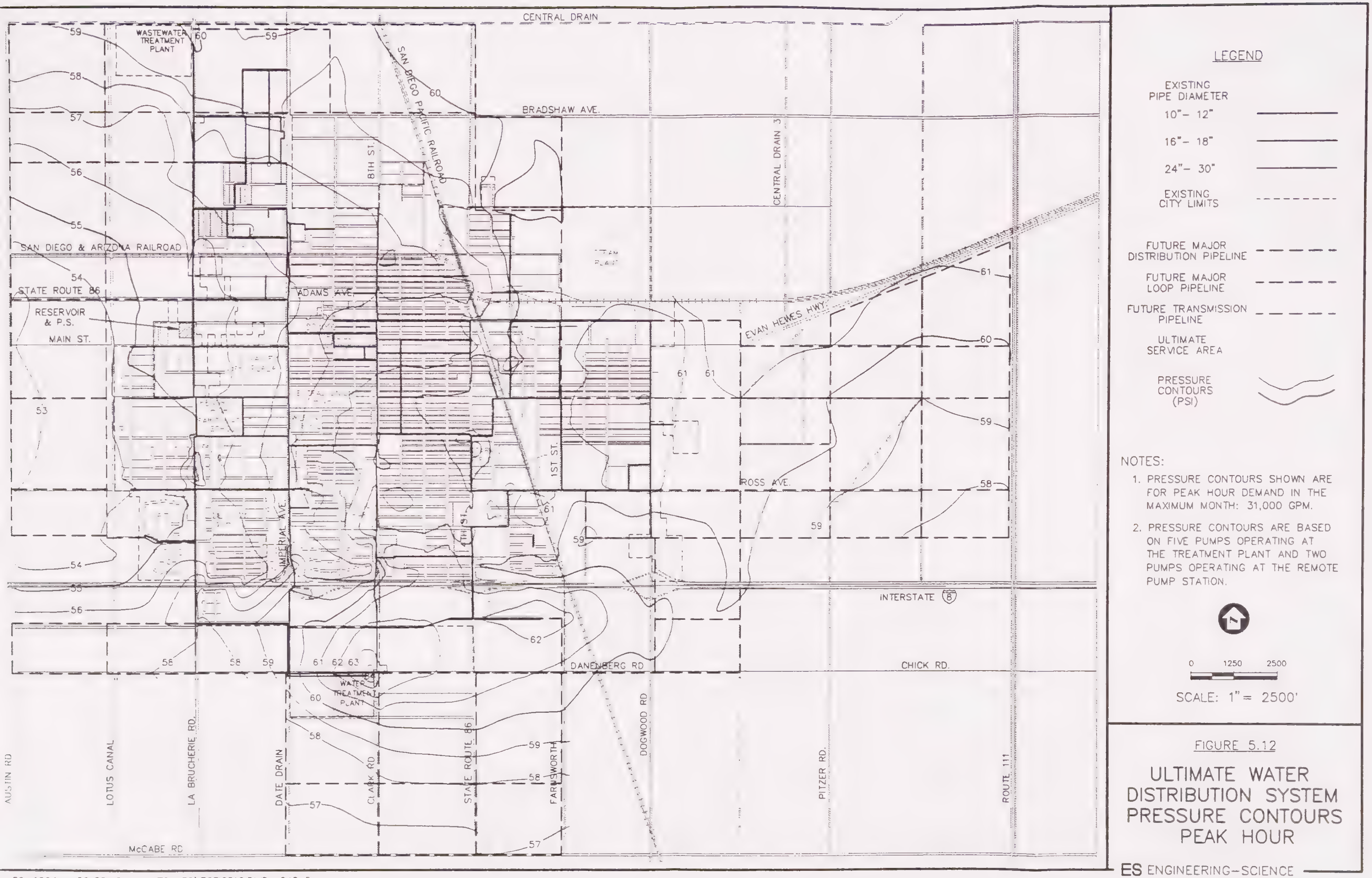
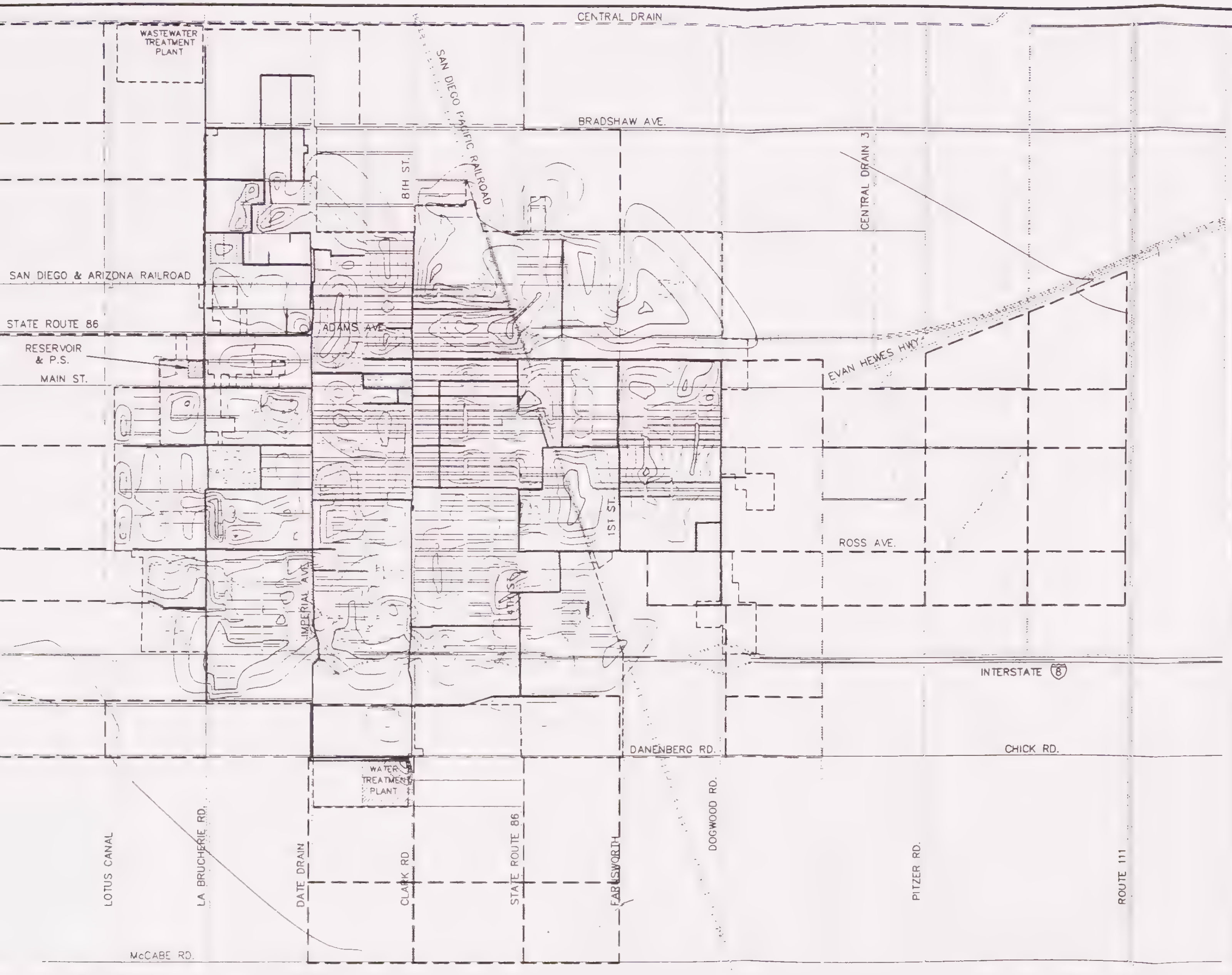


FIGURE 5.12

ULTIMATE WATER DISTRIBUTION SYSTEM PRESSURE CONTOURS PEAK HOUR

ES ENGINEERING-SCIENCE

: 29, 1994 – 20:55:12 724452\ECFIG512.DWG CAD 5



LEGEND

PIPE DIAMETER

10"– 12"

16"– 18"

24"– 30"

CITY LIMITS

FIRE FLOW CONTOURS

1000

2000

3000

4000

5000

0 1250 2500

SCALE: 1" = 2500'

FIGURE 5.13

ULTIMATE WATER DISTRIBUTION SYSTEM AVAILABLE FIRE FLOW CONTOURS

ES ENGINEERING–SCIENCE

1994 – 18:10:53 724452\ECFIG5–13.DWG CAD 5

SECTION 6

SUMMARY OF RECOMMENDED IMPROVEMENTS AND COST ESTIMATES

SECTION 6

SUMMARY OF RECOMMENDED IMPROVEMENTS AND COST ESTIMATES

SUMMARY OF RECOMMENDED IMPROVEMENTS

Recommended improvements to the existing water treatment and distribution system to meet current and ultimate requirements have been developed as described in Sections 4 and 5. Recommended improvements are summarized below.

Water Treatment Plant

The evaluation of the water treatment facilities primarily addressed the raw water storage reservoirs and the filters.

Raw Water Storage

The two existing raw water storage basins presently have 5.7 days of storage at the current average day demand in the maximum demand month. Based on a raw water storage criteria of five days at the maximum month average demand, a third 30 MG storage reservoir is required by the year 1998 and a fourth 30 MG reservoir will be required by the year 2011.

Filter Capacity

On the basis of 24 hour plant operation and a 3.5 gpm/sf maximum loading rate, a fourth filter will be required by the year 1998 to meet maximum day demand. Assuming the fourth filter would have a capacity of 7.4 mgd (half the additional capacity required for ultimate), a fifth filter (7.4 mgd capacity) would be required by the year 2011. It is assumed that expansion of the treated water transfer pump station will be required as part of the duty filter additions to provide for adequate capacity to pump at the filter capacity with one pump available as a standby.

Spare Filter

In order to meet water demand during the summer months with one filter out of service for maintenance or repair, it is recommended that a spare filter be provided for the existing filtration system. The need for a spare filter is considered to be an immediate need. However, providing for a spare filter should be coordinated with providing an additional filter by 1998 to meet filtration capacity requirements. It may be most practical to construct both filters at the same time. It is assumed that the treated water transfer pump station would not need to be expanded to accommodate a spare filter because the spare would not be increasing filtration capacity.

Standby Filter Backwash Pump

The Department of Health Services has indicated that a standby backwash pump is required to meet its requirements for filtration system reliability. Providing a standby backwash pump is therefore recommended as an immediate need. Space in the existing pump room is limited, and addition of the standby pump could be difficult.

Clarifier Capacity

The work of this Master Plan with regards to evaluation of the water treatment plant focused on the capacity requirements of the raw water storage reservoirs and the filtration system. It was noted in Section 4 that increasing the filtration system capacity will also eventually require an increase in clarification capacity. It is therefore noted that additional clarification capacity will be required by the year 2004.

Distribution System

Recommended improvements to the treated water distribution system are described below.

Storage Capacity

The second 5.0 MG storage tank at the remote site will be required by the year 2003. Ultimately, an additional 5.5 MG of storage will be required beginning in the year 2013. At this time it is assumed that the additional 5.5 MG of storage will be provided at the treatment plant site. A remote site on the east side of the City should also be evaluated for the ultimate additional 5.5 MG storage. It is assumed that the additional ultimate 5.5 MG storage will be provided in the form of above-grade steel tanks.

Duty Pumping Capacity

The additional 3,500 gpm pump at the remote pump station will be required by the year 1999 to meet peak hourly demand. An additional 4,000 gpm pump at the treatment plant site will be required by the year 2003 to meet maximum day pumping requirements. From the year 2012 to ultimate, an additional 9,000 gpm in pumping capacity will be required. At least 5,000 gpm of this additional ultimate capacity must be provided at the treatment plant site by 2013 to meet ultimate maximum day demand. The remaining 4,00 gpm capacity could be provided at a remote site with the additional ultimate 5.5 MG storage capacity. For the purposes of this Master Plan, it is assumed that the total 9,000 gpm ultimate additional pumping capacity will be provided at the treatment plant site. However, providing 4,000 gpm of the additional ultimate capacity at a remote site with the additional ultimate storage capacity of 5.5 MG should be evaluated.

Standby Pumping Capacity

The 100 hp constant speed standby pump at the water treatment plant needs to be upgraded to a 200 hp, variable speed pump to provide adequate standby capacity at the 60 psi distribution system operating pressure. Since the standby pump cannot provide adequate capacity to meet present demands with one pump at the plant out of service, this upgrade is considered an immediate need.

Pipelines

Five pipeline segments are recommended to improve the existing water distribution system: (1) a 27-inch (or 30-inch), 2,400 ft transmission main in La Brucherie Road from Wake Avenue to Ocotillo Drive; (2) an 18-inch, 1,300 ft transmission main parallel to the existing 18-inch main along the Date Drain from Danenburg Road to Wake Avenue; (3) a 24-inch, 2,500 ft line parallel to the existing 18-inch main in Danenburg Road from the treatment plant to the Date Drain; (4) a 12-inch, 325 ft connection line in Ross Avenue from 2nd Street to 1st Street; and (5) a 12-inch, 1,700 ft main in 4th Street from Ross Avenue to Hamilton Avenue. Pipelines 1 through 3 are recommended to provide a direct supply to the south-west and west areas of the City. Pipeline 4 is recommended to provide a more direct supply to the south-east area of the City and pipeline 5 is recommended to improve looping in the east-central area. The recommended pipeline installations are not immediate needs. However, as the south-west and west areas develop, the need for pipelines 1 through 3 will increase. It is recommended that the pipeline improvements be phased in over an period such as five years.

Pipelines required for the ultimate service area ~~have~~ been developed on a conceptual level. These pipelines will be installed as required to serve future growth. Therefore, phasing of the future pipeline system will depend on the growth patterns in the ultimate service area.

COST ESTIMATES

Cost estimates are conceptual level and include a contingency and allowances for engineering. Costs are in present dollars and are referenced to the current ENR Construction Cost Index for Los Angeles of 6510. Cost estimates for the recommended treatment, treated water storage, and distribution improvements are presented in Table 6.1. Cost estimates for recommended pipeline improvements for the existing distribution system are presented in Table 6.2. Table 6.3 itemizes the additional pipelines required for the ultimate distribution system and provides cost estimates for the lines. Additional information regarding the basis of the cost estimates is provided below.

Raw Water Storage Reservoirs

A unit cost of $0.05/gal was used for construction of the earthen-levee, asphalt paved raw water storage basins. The total estimated cost for each reservoir includes 20 percent contingency and 10 percent for engineering.

Filters

A cost of $1,000,000 was used for construction of cast-in-place concrete filters based on typical construction cost curves for gravity filters (Reference 12). For the additional duty filters, $150,000 construction cost was added to upgrade the treated water transfer pump station. The cost estimate assumes upgrade of the backwash facilities is not required for the additional filters. Filter costs include 20 percent contingency and 15 percent for engineering.

Standby Backwash Pump

The estimated cost for providing a standby backwash pump is based on installing the pump in the existing pump room and includes piping modifications and electrical system improvements. The estimated cost for the standby backwash pump includes 20 percent contingency and 15 percent for engineering.

Treated Water Storage Tanks

A unit cost of $0.15/gallon was use for construction of the tank at the existing remote site. A unit cost of $0.20 was used for all other tank construction to account for required site work. The estimated costs for the treated water storage includes 20 percent contingency and 10 percent for engineering for the additional tank at the remote site and 15 percent for engineering for other storage tanks.

Distribution Pumping

The remote pump station was designed to allow for addition of a third pump. The fourth duty pump at the treatment plant could be accommodated by modifying the surge control valve bypass; however, future expansion of the treatment plant pumping capacity beyond a fourth duty pump could not be accommodated by the existing facilities and it is assumed that a new pump station would be required. Cost estimates for new pumping facilities are based on typical cost curves (Reference 13) with 20 percent contingency and 15 percent for engineering.

Water Treatment Plant Standby Pump Upgrade

The cost estimate for upgrading the standby distribution pump at the water treatment plant is based on replacing the pump and motor and providing an additional VFD and includes 20 percent contingency and 10 percent for engineering.

Pipelines

Unit costs for pipeline construction used in Tables 6.2 and 6.3 are based on recent pipeline construction costs in the City. Unit costs include street resurfacing, valves, fittings, appurtenances, 15 percent contingency, and 10 percent engineering/inspection and are based on an average depth of 5 ft.

PHASING OF IMPROVEMENTS

Treatment, storage, and pumping improvements identified as immediate needs are addition of a spare filter, a standby filter backwash pump, and upgrading of the standby pump at the treatment plant water distribution pump station. Treatment, storage, and pumping improvements required by the year 1998 are addition of a raw water storage pond, a fourth duty filter, and upgrading the capacity of the treated water transfer pump station. Installation of a third pump at the remote pump station is projected to be required by 1999. Phasing of recommended pipeline improvements to the existing system to improve flow distribution are proposed to be phased in over a five year period. Table 6.4 shows suggested phasing of the recommended improvements. It was assumed for Table 6.4 that improvements required beyond 1999 (five years) will be implemented in the projected year required.

Phasing of pipeline additions to the system to serve future development will be determined by growth patterns and will be planned in conjunction with specific development planning. Therefore, phasing of distribution system pipelines for future service areas is not addressed in Table 6.4.

Table 6.1

Estimated Costs for Recommended Treatment, Treated Water Storage, and Distribution Pumping Improvements

Year Required	Recommended Improvement	Estimated Cost[a] ($1,000)
	Treatment Facilities[b]	
1995	Standby filter backwash pump	$200
	Spare filter	$1,400
1998	30 MG raw water storage reservoir	$2,100
	7.4 mgd filter	$1,600 [c]
2011	30 MG raw water storage reservoir	$2,100
	7.4 mgd filter	$1,600 [c]
	Total Treatment	$9,000
	Treated Water Storage Facilities	
2003	5.0 MG remote storage tank	$1,000
2013	5.5 MG treated water storage	$1,500
	Total Treated Water Storage	$2,500
	Distribution Pumping Facilities	
1995	WTP standby pump upgrade	$250
1999	3,500 gpm remote pump	$100
2005	4,000 gpm WTP distribution pump	$250
2012	5,000 gpm WTP distribution pump capacity	$700
2017	4,000 gpm distribution pump capacity	$500
	Total Distribution Pumping	$1,800
	Total Treatment, Storage, and Pumping	$13,300

[a] ENR Construction Cost Index 6510, April 1994.

[b] Improvements to water treatment facilities address raw water storage and filtration system only.

[c] Includes cost to upgrade treated water transfer pump station.

Table 6.2

Estimated Costs for Existing Water Distribution System Pipeline Improvements

No.	Location	Description	Diameter (inch)	Length (feet)	Unit[a] Cost ($/ft)	Total Cost ($1,000)
1	La Brucherie Road	Wake Avenue to Ocotillo Drive	27 [b]	2,400	$100	$240
		I-8 Crossing		500	$500	$250
2	Date Drain	Danenburg Road to Wake Avenue	18	1,300	$75	$100
3	Danenburg Road	Treatment Plant to Date Drain	24	2,500	$90	$225
4	Ross Avenue	1st Street to 2nd Street	12	325	$60	$20
5	4th Street	Ross Avenue to Hamilton Avenue	12	1,700	$60	$100
	Total Pipeline Improvements					$935

[a] ENR Construction Cost Index 6510, April 1994.

[b] Or 30-inch diameter.

Table 6.3

Estimated Costs for Future Water Distribution System Pipelines

Development Area	Diameter (inch)	Length (feet)	Unit[a] Cost ($/ft)	Total Cost ($1,000)
South	30	18,600	$110	$2,050
	27	1,500	$100	$150
	18	11,000	$75	$830
	12	47,700	$60	$2,860
West	24	14,600	$90	$1,310
	12	38,500	$60	$2,310
North	24	14,100	$90	$1,270
	18	7,300	$75	$550
	12	14,100	$60	$850
East	30	300	$110	$30
	27	8,000	$100	$800
	18	2,300	$75	$170
	12	42,600	$60	$2,560
Total for Future Water Distribution Pipelines				$15,740

[a] ENR Construction Cost Index 6510, April 1994.

Table 6.4

Suggested Phasing for Recommended Improvements

Fiscal Year	Recommended Improvement (year required)[a]	Estimated[b,c] Cost ($1,000)
1995/96	Standby filter backwash pump (1995)	$200
	WTP standby pump upgrade (1995)	$250
	La Brucherie Road 27-inch main	$490
	Date Drain/Danenburg Rd. parallel mains	$325
	Total 1995/96	$1,265
1996/97	Spare filter (1995)	$1,400
	7.4 mgd filter (1998)	$1,600
	Total 1996/97	$3,000 [d]
1997/98	30 MG raw water storage reservoir (1998)	$2,100
1998/99	3,500 gpm remote pump (1999)	$100
	Ross Avenue 12-inch connection	$20
	4th Street 12-inch main	$100
	Total 1998/99	$220
2003	5.0 MG remote storage tank (2003)	$1,000
2005	4,000 gpm WTP distribution pump (2005)	$250
2011	30 MG raw water storage reservoir (2011)	$2,100
	7.4 mgd filter (2011)	$1,600
	Total 2011	$3,700
2012	5,000 gpm WTP distribution capacity (2012)	$700
2013	5.5 MG treated water storage (2013)	$1,500
2017	4,000 gpm distribution pump capacity	$500
	Total Improvements	$14,200

[a] Improvements to water treatment facilities address raw water storage and filtration system only.

[b] ENR Construction Cost Index 6510, April 1994.

[c] Refer to Tables 6.1 through 6.3 for cost estimates.

[d] Constructing both filters together could reduce cost; cost estimate is for separate construction.

APPENDIX A

REFERENCES

REFERENCES

1. Lyon Engineers, Master Water Plan 1982 Update, prepared for the City of El Centro, 1982.
2. Engineering-Science, Inc., Alternatives Study: Proposed Steel Reservoir and Pump Station, prepared for the City of El Centro, March 1988.
3. Brian F. Mooney Associates, City of El Centro, California, 1990 General Plan March 15, 1989.
4. Jerry H. McLees, Brian H. Mooney Associates, Personal Communication, May 1994.
5. Hofman Planning Associates, Draft Service Area Plan for Proposed Annexation, March 11, 1994.
6. City of El Centro Planning Department, Personal Communication, March 1994.
7. Linual White, City of El Centro Fire Marshal, Personal Communication, June 1994.
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9. Toby Roy, State Department of Health Services, Personal Communication, June 1994.
10. Paul Steward, City of El Centro, Personal Communication, June 1994.
11. California Domestic Water Quality and Monitoring Regulations, Title 22, Section 64660, Operating Criteria, 1990.
12. James M. Montgomery, Consulting Engineers, Inc., Water Treatment Principles and Design, 1985.
13. Robert L. Sanks, et al, Pumping Station Design, 1989.

APPENDIX B

MODEL CALIBRATION DATA

Table B.1

Fire Hydrant Pressure Test and Calibration

Location	Discharge Pressure(1)	System Pressure (2)	Recorder Pressure (3)	Simulation Pressure
Water Treatment Plant - High Flow = 8,850 gpm, Pump #1 Speed = 100%, Pump #2 Speed = 99.7%				
9th & Broadway	55.90	59.85	59.0	57.55 (4)
6th & Euclid	55.90	59.85	59.0	57.60 (4)
6th & Hamilton	55.90	59.85	58.0	57.52 (4)
23rd & Ocotillo	55.90	59.85	55.0	52.03 (4)
Aurora btw. 3rd & 4th	56.48	59.30	59.75	58.78
Ross & McCullum	56.48	59.30	58.0	57.16
19th & Villa	56.48	59.30	56.75	56.93
Main btw. Lotus & 23rd	56.48	59.30	55.75	54.62
3rd & Commercial	56.48	59.30	59.30	59.70
Remote Pump Station - High Flow = 8,750 gpm, Pump #1 Speed = 100%, Pump #2 Speed = 100%				
9th & Broadway	57.90	49.75	n/a	49.38 (5)
6th & Euclid	57.90	49.75	n/a	49.87 (5)
6th & Hamilton	57.90	49.75	n/a	47.66 (5)
23rd & Ocotillo	57.90	49.75	n/a	44.00 (5)
Aurora btw. 3rd & 4th	57.90	49.75	49.5	46.24
Ross & McCullum	57.90	49.75	50.5	48.20
19th & Villa	57.90	49.75	55.0	53.74
Main btw. Lotus & 23rd	57.90	49.75	51.5	51.89
3rd & Commercial	57.90	49.75	49.75	50.07
Water Treatment Plant - Low Flow = 2,760 gpm, Pump #1 Speed = 86%				
9th & Broadway	55.90	59.85	59.0	58.80 (4)
6th & Euclid	55.90	59.85	59.0	59.59 (4)
6th & Hamilton	55.90	59.85	58.0	57.74 (4)
23rd & Ocotillo	55.90	59.85	55.0	52.69 (4)
Aurora btw. 3rd & 4th	51.50	59.70	58.0	60.68
Ross & McCullum	51.50	59.70	59.25	62.40
19th & Villa	51.50	59.70	58.25	62.76
Main btw. Lotus & 23rd	51.50	59.70	57.00	59.81
3rd & Commercial	51.50	59.70	59.70	60.20

Table B.1 (continued)

Location	Discharge Pressure(1)	System Pressure (2)	Recorder Pressure (3)	Simulation Pressure
Remote Pump Station - Low Flow = 2,760 gpm, Pump #1 Speed = 87				
9th & Broadway	52.15	60.28	59.25	62.7 (4)
6th & Euclid	52.15	60.28	60.0	63.35 (4)
6th & Hamilton	52.15	60.28	58.0	61.68 (4)
23rd & Ocotillo	52.15	60.28	54.25	56.64 (4)
Aurora btw. 3rd & 4th	56.10	59.63	57.25	46.24
Ross & McCullum	56.10	59.63	60.0	48.20
19th & Villa	56.10	59.63	58.75	53.74
Main btw. Lotus & 23rd	56.10	59.63	58.25	51.89
3rd & Commercial	56.10	59.63	n/a	50.07

Notes:

1. Discharge pressure is the pressure in the discharge line downstream of the operating pumps as read from the motor control center at the main pump station.
2. System pressure is the pressure at 3rd and Commercial.
3. Recorder pressure is the pressure that city staff recorded in the field during the hydrant flow tests under the conditions outlined.
4. Flow conditions slightly different from those outlined.
5. Hydrant Flow Test conducted at a flow rate of approximately 7750 GPM.

APPENDIX C

HYDRAULIC MODELING RESULTS

(BOUND SEPARATELY)